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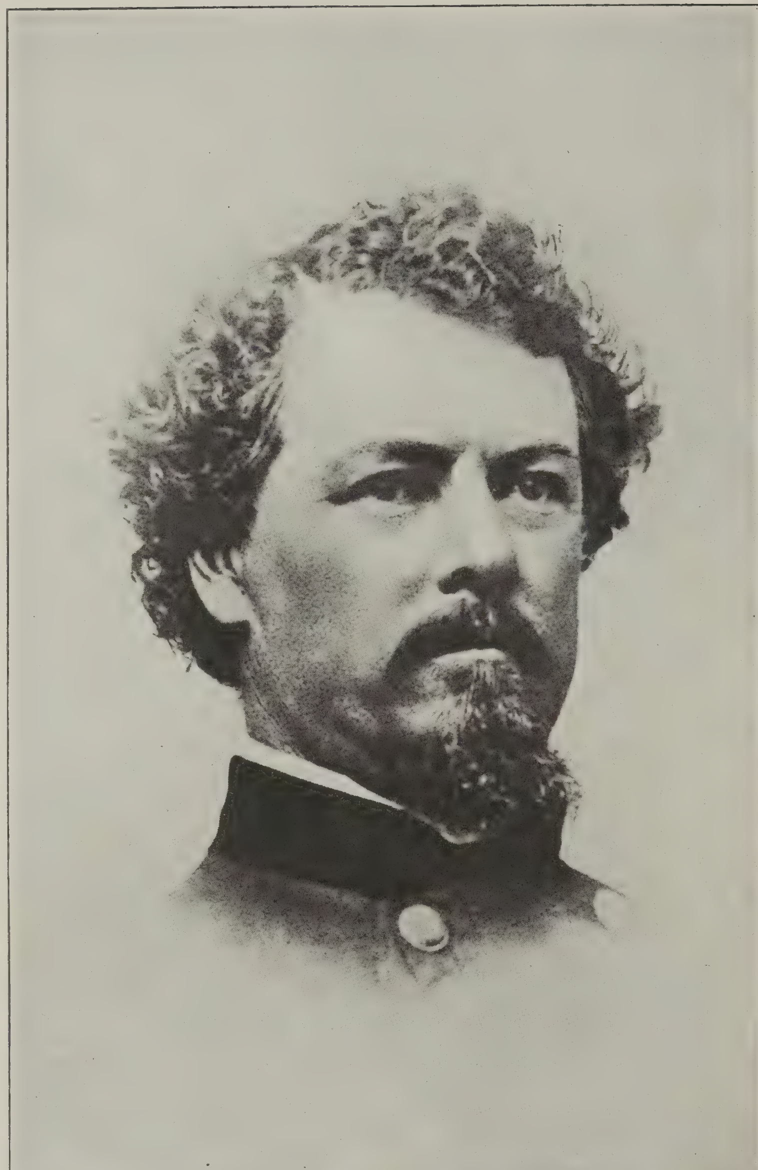

BRIG. GEN. HORATIO GOVERNEUR WRIGHT

CHIEF OF ENGINEERS, UNITED STATES ARMY

1879-1884

BORN 1820—DIED 1899

Los Angeles Harbor

BY

Capt. AMOS A. FRIES
Corps of Engineers; Member American Society of Civil Engineers

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Just when Los Angeles Harbor (formerly known as San Pedro Harbor) was discovered is not known, but as Santa Catalina Island, 20 miles distant, was discovered in 1542, it is assumed that Los Angeles Harbor was visited about the same time.

Two hundred and thirty-nine years later (in 1781) Los Angeles was founded by a small colony of monks from the San Gabriel Mission, 12 miles distant. The San Gabriel Mission itself was only ten years old at the time. These two missions were part of a chain extending from San Diego to San Francisco, a distance of 600 miles. Although these missions were founded in fertile valleys near fresh water streams they were, wherever possible, near harbors or protected anchorages. For example, there were missions at San Diego, Los Angeles, Santa Barbara, Monterey, and San Francisco.

During the following fifty-four years little definite is known of of the harbor's history. Dana mentions it, in his "Two Years Before the Mast," as a place where hides were rolled down the bluffs to small boats, in which they were carried to ships anchored in the bay. The real history of the harbor began in 1846, when the Mexican government confirmed a grant made in 1827 of several thousand acres of land known as the Palos Verde Rancho. This rancho followed the San Pedro Bay shore on the west, and the Mexican government, with a foresight seldom exhibited by our Government, reserved for public use about 1,400 feet of shore line with 42 acres of land back of it in the following terms:

4th. They shall leave free *on the beach of San Pedro,* five hundred varas square, to the four cardinal points, upon which houses may be built by persons who may obtain permission to do so; they shall not be permitted to prevent the use of the water and pasture by persons engaged in traffic with oxen or horses to the Port of San Pedro.

From here the business of the port was carried on until about the time of the Civil War, when the landing place was shifted to the inner bay at Wilmington. This shift did away with some 3 miles of wagon haul over hilly roads, with little or no added cost of lighterage. These places are shown on Plate I.

It was during the Civil War, and the Indian wars immediately following, that the harbor attained its first real importance. Yuma, Ariz., on the Colorado River, 120 miles from its mouth, was the starting place for supply trains and troops operating against the Indians in the interiors of New Mexico and Arizona. There were only two ways of reaching Yuma. The first was to go by boat around Lower California and up the gulf of that name to the Colorado River, where cargoes were transferred to river steamers running directly to Yuma. The second was to land at Los Angeles Harbor and cover the 250 miles overland to Yuma by wagon train. For various reasons a great deal of business was carried on by the latter route.

Following the Civil War and the lull in the gold hunting fever in the Sacramento Valley, Los Angeles and the surrounding country began to settle up with Americans who were attracted by the soil and climate. There being no transcontinental railroads, the need of harbor facilities superior to those naturally existing was felt so strongly that, in 1871, the Government began the work of building jetties to straighten and deepen the entrance to the inner harbor leading to the landing place at Wilmington.

As this marks the beginning of what is destined to be one of the great harbors of the world, it is worth while to review briefly the natural conditions of this harbor and the reasons why its development was undertaken and why it will unquestionably be extended in the future until the harbor becomes, as stated above, one of the great harbors of the world.

The western coast line of the United States is approximately 1,500 miles in length. Due to the nearness of the Coast Range of mountains to the shore, there are only four localities where harbors and commercial centers may be economically developed. Beginning at the north, the first is the Puget Sound country in Washington. Next southward is the Columbia River, Oregon and Washington. From the mouth of the Columbia River there is no place capable of large economic development until San Francisco Bay is reached, nearly 700 miles distant. Again going south, the first locality reached capable of commercial development is in the

vicinity of Los Angeles, 500 miles distant. As a matter of fact, so far as tributary country is concerned, the entire coast may be divided into three commercial districts: the Puget Sound and Columbia practically forming one, San Francisco Bay another,

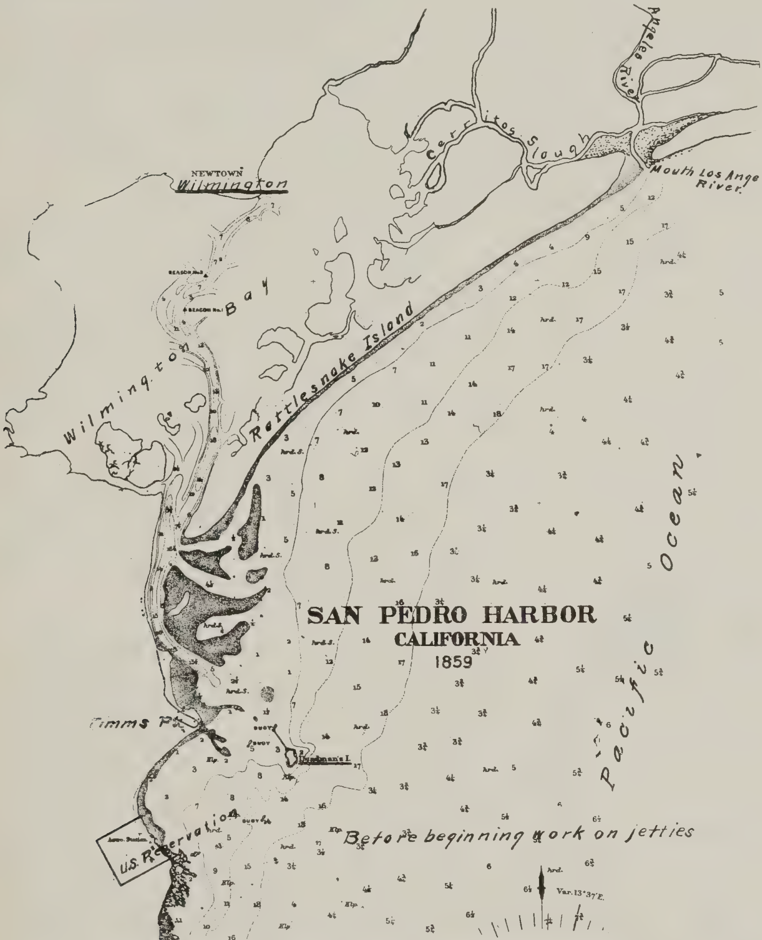

Plate I. From United States Geological Survey chart.

and the hundred miles between Los Angeles and San Diego the third.

Not only does the Coast Range of mountains limit the commercial centers to the above, but the passes to the east through the Sierra Nevada and Rocky mountains do the same.

Popularly, a harbor is a great bay of water, such as San Fran-

cisco and New York, with narrow wooden wharves projecting into it, and few people can conceive of a harbor existing or being made where no such bay exists. Los Angeles, however, is demonstrating that such a bay is not necessary and, in fact, that the nearness of a fine bay deficient in some of the other essential features of a great harbor will not prevent the growth of another harbor less favored by nature, nor seriously reduce the amount of commerce coming into it. If this were not the case, the magnificent bay at San Diego, 100 miles south of Los Angeles Harbor, would have attracted to it all the ocean commerce south of San Francisco.

San Diego Bay is a good natural harbor of large extent. In its original condition there was ample depth of water inside for the largest modern ships, but the draft of vessels entering was limited to 21 feet by a bar at the entrance. The depth on the bar is rapidly being increased, but the harbor is seriously handicapped by the fact that it is on the extreme southwest border of the United States with no easy pass across the mountains to the East, except that near Los Angeles, without going some distance below the Mexican line.

Furthermore, the agricultural lands back of the bay are very much smaller in extent than those near Los Angeles, while closely surrounding the bay there are bluffs from 400 to 800 feet high that must be surmounted within a very few miles after leaving the water front.

Los Angeles, on the other hand, has a great advantage in all these respects. Its harbor is the nearest one to the passes over the mountains east and north, and is apprcached in every direction except the extreme west by grades of only about 10 feet to the mile, extending many miles into the back country. The rich agricultural country immediately surrounding it has been rapidly developed, and with this development the city has grown like magic.

Indeed, a land-locked bay and deep water alone will never make a great harbor for commerce. The essential requirements of such a harbor are: first, a sheltered anchorage accessible in any weather; second, quiet water for wharves and docks; third, land just a trifle above tide water for factory sites, railroads, and all the machinery for commerce and manufacturing; fourth, and important above every other consideration, the harbor must be surrounded by a rich and populous country which in its turn is easily accessible to all inland lines of travel as well as to the harbor itself.

Then, too, the nearness of a harbor to other important harbors

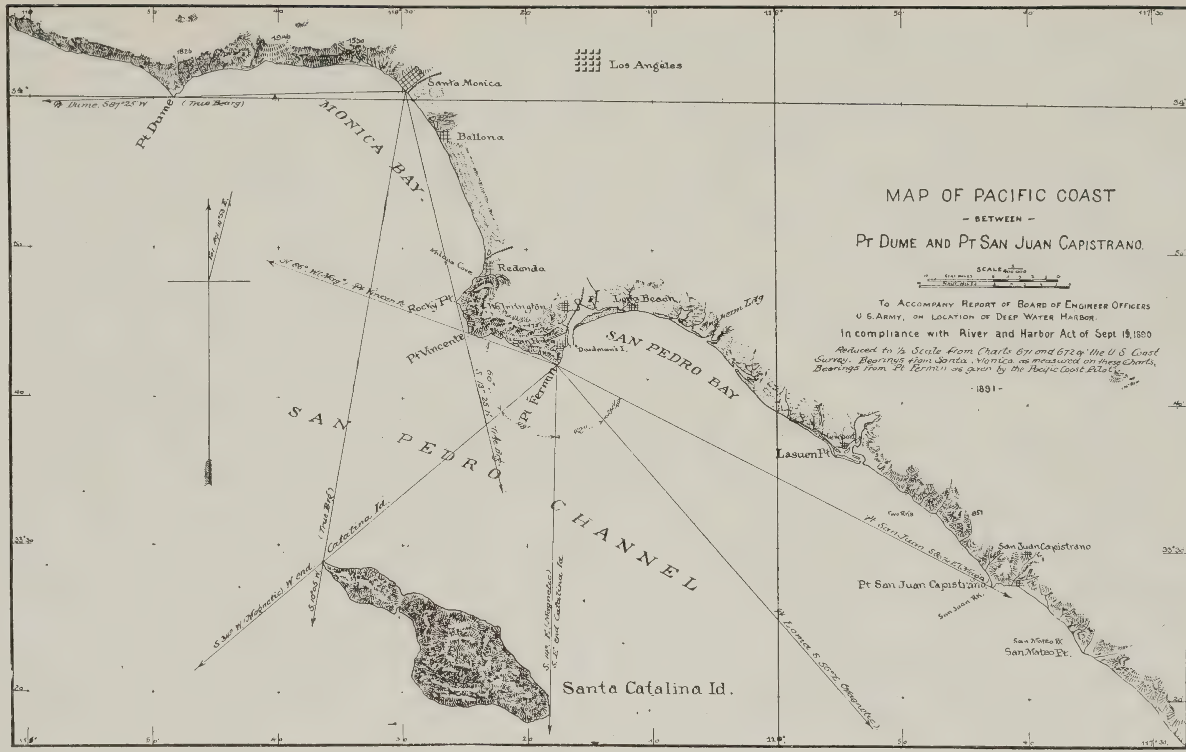

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Plate II. Showing how Los Angeles Harbor was naturally protected on the south and southwest by Santa Catalina Island; on the west by Point Fermin, and on the east and southeast by the mainland, leaving only a 40-degree angle to the south unprotected.

and commercial centers determines largely how great it can really become. If too near, the trade that might go through the one harbor will be distributed among several.

Applying these tests of a great harbor to the one at Los Angeles, it is found that the latter fulfills them to an exceptional degree, as follows: First. The anchorage in San Pedro Bay, even before the construction of the breakwater, was safer than other nearby localities for three reasons: (a) the bottom is such that anchors hold exceptionally well; (b) Catalina Island, 18 to 25 miles distant, shelters it from south to southwest storms, and (c) the more westerly storms are cut off by Point Fermin. This point is simply the continuation to water level of San Pedro Hill, 1,475 feet high.

At the entrance to Los Angeles Inner Harbor (see Plate I) there is formed a half-moon bay, the distance from Point Fermin to Deadmans Island being about 1½ miles. This formation, together with the existence of the inner harbor, is what has led every board of engineers who have reported upon the best place to build a breakwater for a harbor "for commerce and of refuge" in Southern California to choose San Pedro Bay.

As to the second consideration, viz, quiet water for wharves and docks, it should be noted that the Los Angeles and San Gabriel rivers in ages past wandered over the territory covered by the city of Los Angeles, as well as over the land between Los Angeles and the sea, discharging their waters into the ocean now in one place and then in another. When the white man first came to this locality, the Los Angeles River, augmented by the San Gabriel, was occupying its present bed and discharging through the present mouth at Long Beach, some 6 miles northeast of Point Fermin. However, in 1867, the San Gabriel changed its outlet to Alamitos Bay, a few miles to the east, but returned to Long Beach in 1910. Between the Long Beach mouth and San Pedro Hill there is an island of sand formed by the wind and waves, formerly called Rattlesnake Island, but now known as Terminal Island. Between this island and the mainland to the north (½ to 1½ miles distant) is Wilmington Bay with an area of about 1,400 acres. Originally, this bay was shallow in most places, with deeper channels running in all directions, one of which connecting with the mouth of the Los Angeles River formed the northern boundary of Terminal Island. (See Plate I.)

In addition to Wilmington Bay proper there are hundreds of acres of marsh and tide land between its eastern boundary and Long

Beach, and in neither area are there any rocks and almost no hard material, hence dredging is cheap and easy. It is in this bay and the adjacent marsh and tide land that the real commercial harbor is being developed and extended by dredging ample basins and channels which, being completely shut in by land, have absolutely quiet water.

As to the third requirement for a great harbor, viz, land just a trifle above tide water for factories, warehouses, etc., the situation could hardly be bettered. Everywhere stretching away toward the interior from the harbor, except near the entrance on the San Pedro side, the land is very flat for several miles and just a few feet above high tide. This will enable railroads to build in every direction cheaply and with no sensible grades, and for miles there can be railroad yards and manufacturing plants of every description, all of which can be reached by channels dug through the soft material of Wilmington Bay.

As for the fourth condition, viz, that a harbor must be surrounded by a rich and populous country, Los Angeles Harbor stands preeminent in the Pacific southwest. There is to-day within a radius of 60 miles of the harbor a rapidly growing population of three-quarters of a million. Yet its agricultural development is only just beginning, because until within the past twenty-five years the whole country was held in very large ranches and put to no use except for grazing and the production of small grain.

Finally, as to accessibility—the fifth condition for a great harbor—Los Angeles is located opposite the best passes across the mountains to the east and in a central position regarding trade up and down the coast and into the interior of California itself. It is the nearest point to the great mineral belt comprising the States of Utah, Nevada, New Mexico, Arizona, and Colorado, not to mention Mexico. It is also the nearest important commercial center to the main route of travel between the Panama Canal and the Orient. It is nearer Salt Lake City and all points south and east of that city than is San Francisco itself, and as for the cotton belt along the Gulf States, from Texas to South Carolina, it is generally nearer by from 300 to 400 miles.

As heretofore stated and as shown on Plate I, the Los Angeles anchorage was naturally only an open roadstead protected on the south and southwest from the long surges of the Pacific by Catalina Island, 18 to 25 miles distant, and on the west by San Pedro Hill terminating in Point Fermin. About 1½ miles north and ⅞ of a

mile east of Point Fermin is Deadmans Island, rising to an elevation of 60 feet above the water with an area of less than 2 acres. From Point Fermin the shore line runs nearly due north for 1½ miles and then turns abruptly to the east for a distance of about ½ mile to Timms Point. Here, again, it turns more than 90 degrees to the northwestward. From Timms Point to Deadmans Island, ½ mile southeast, the water was originally only about 2 feet deep at low tide.

From Deadmans Island north for 1¼ miles there was shallow water, usually from 1 to 3 feet deep at low tide but with crooked shifting channels with from 3 to 6 feet of water. (See Plate I.) This was the main tidal entrance to Wilmington Bay and the entrance for boats and lighters prior to the beginning of improvements by the Government in 1871.

Rattlesnake Island, now called Terminal Island, limited this entrance on the north and extended in a general easterly direction 4 miles to the mouth of the Los Angeles River at the present town of Long Beach. This island was low and sandy and bounded on the north by a tortuous channel about 100 feet wide and from 3 to 6 feet deep, known as Cerritos Slough. Wilmington Bay, formed by Rattlesnake Island on the south and the mainland on the west, was composed partly of tide land covered at high tide and partly submerged land with water 1 to 3 feet deep at low tide —the whole threaded with deeper channels. The main channel from the sea extended nearly due north to high ground near Wilmington, a distance of 3 miles from Deadmans Island. Here it turned eastward for about 1 mile and then, sweeping southward, joined with Cerritos Slough.

When the United States acquired California by the Treaty of Guadalupe Hidalgo, in 1848, land titles were in a sadly mixed state, owing to overlapping grants and many fraudulent claims. In order to settle these titles, a special commission was appointed in 1851 to determine the proper boundaries of grants and the rightful owners thereof. The District Federal Court for the Southern District of California passed finally on all of these claims, which, after being surveyed and described by metes and bounds, were patented to the owners by the United States.

When the San Pedro grant was surveyed, the usual outside bounding lines were described, after which that part of Wilmington Bay coming within these lines was specifically excepted as

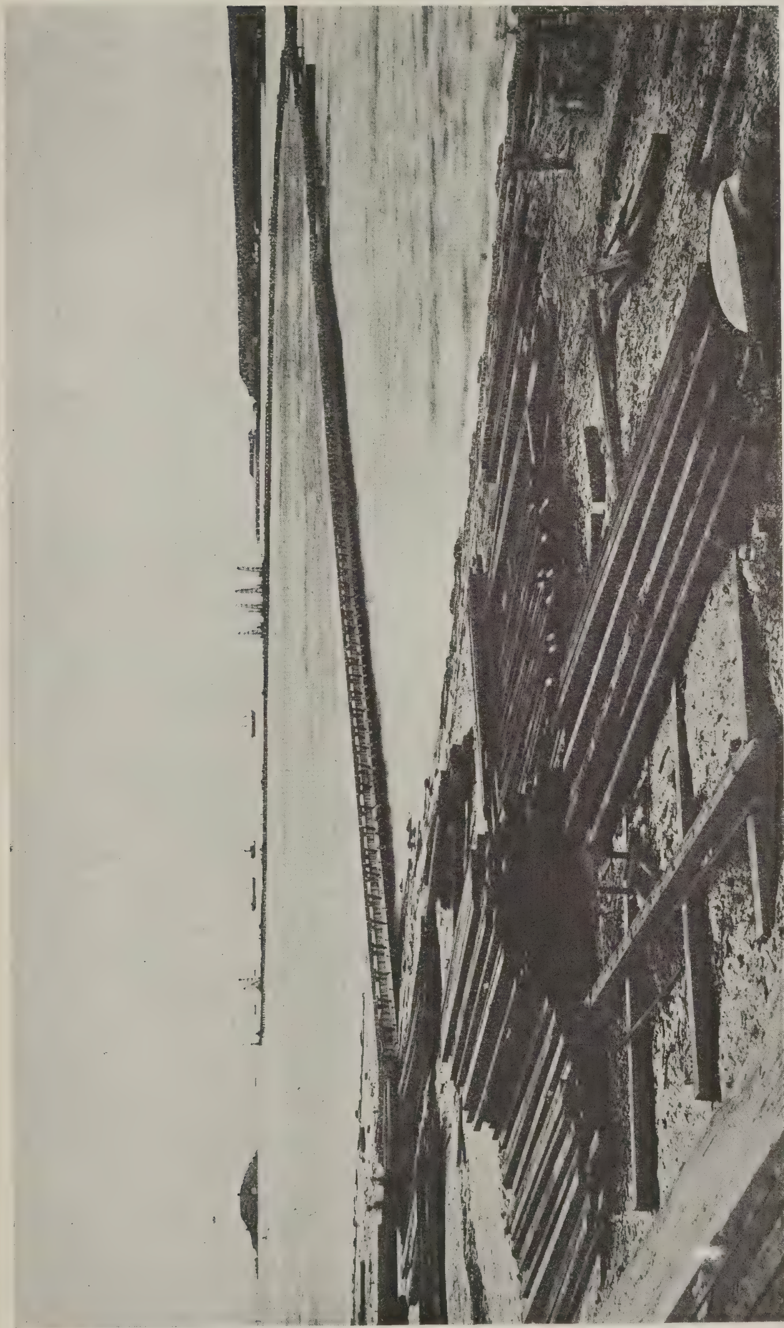

Fig. 1. The entrance to Los Angeles Inner Harbor in August, 1872, showing the east jetty and the pile-drivers at work on the wooden sections of that jetty. Note Deadmans Island on the left and Timms Point on the right.

navigable waters of the United States in the following terms:

> Excepting, reserving, and excluding from said tract, as thus surveyed, that portion thereof covered by the navigable waters of the inner bay of San Pedro, and which are included within the following described lines, to-wit: * * *

No effort will be made to trace the public's fight against private and corporate greed, first to get a harbor at all and then to get it under public control, as that is a long, intricate story, although a highly instructive one.

Rattlesnake Island was unquestionably only the development of the bar at the mouth of the Los Angeles River, beginning ages ago when Wilmington Bay extended many miles inland. As the swirling, silt-bearing floods of the Los Angeles and San Gabriel rivers were halted by the ocean, the silt was dropped, only to be later picked up and tossed shoreward by the breakers driven before strong south winds. In a similar way the energy of the sea forced the river westward to the shelter of San Pedro Hill, though it succeeded in maintaining a slight channel at the eastern end of Rattlesnake Island.

Notwithstanding the Long Beach channel, the main entrance to the bay remained under the shelter of San Pedro Hill north of Deadmans Island, and here the mass of the tide entered and receded.

Apparently the westward growth of Rattlesnake Island was retarded and turned southward by the ebb tide which, impinging against the San Pedro bluffs, was thrown eastward against the extreme westerly point of Rattlesnake Island, which thus became a natural jetty. Such were the conditions when the Government undertook the improvement of the harbor.

The first work was directed to extending Rattlesnake Island as a jetty on a curve to Deadmans Island, 1¼ miles south, in order to confine the ebb tide and cause it to deepen the entrance, while at the same time it straightened the main channel.

Inasmuch as the 3-fathom curve was nearly 1 mile out in the bay beyond Timms Point, a western jetty was necessary to confine the waters on that side if any considerable depth was to be attained by the scouring action of the tides.

The eastern jetty, begun in 1871, was completed ten years later and was so successful that where originally the depth across the bar between Deadmans Island and Timms Point was only 2 feet,

there was after the completion of the jetty a good channel close to Deadmans Island, with a least depth of 10 feet.

The construction of the jetties (breakwaters they were called at first) at the entrance to Los Angeles Harbor was not accomplished without the vicissitudes and setbacks common to engineering work in new localities and under strange conditions.

The east jetty connecting Rattlesnake and Deadmans islands was about 6,700 feet in length, and during construction was divided into three sections. The first section, 3,700 feet long, began at Rattlesnake Island and consisted of a line of double sheet piles well braced and rising about 1 foot above high water. The next 1,000 feet consisted of two parallel rows of 12-inch sheet piling 10 feet apart, strongly braced and partially filled with brush, stone, gravel, and sand. (See Fig. 1.)

The 2,000 feet next to Deadmans Island is a rubble mound, built up by simply depositing the rock on the sand and carrying it up until it was several feet above high water. (See Fig. 2.) A number of breaks, especially along the line of the old channels, occurred in these pile jetties and in nearly all places it was found necessary to strengthen them very considerably from what was originally planned. For this strengthening, rock was generally used.

It was assumed that sand would accumulate on the seaward side of the east jetty in quantities, and soon enough to make the wooden jetty permanent without further repairs. This, however, did not prove true, and spurs or groins were built out to intercept the sand. These, too, proved only partially successful. Planting of trees and grass was also tried but with very little success, and from time to time repairs were necessary. The last repairs made were in 1903, and consisted in dumping stone to replace woodwork and to keep the waves from breaking across the jetty. The first and second sections, due to differences in construction, were later referred to as the single work and double work, depending upon whether there was a single or a double row of piling. (See Fig. 2.)

In some places the single work was finally made of solid timber bulkheads, consisting of timbers 12 to 18 inches wide placed on top of one another and nailed or drift-bolted together. All but about 1,000 feet of this east jetty is now protected by an outer bulkhead and about 1,000 feet of solid fill. The type and methods

followed in constructing the east jetty are fairly well shown in the accompanying illustration (Fig. 1).

As stated at the beginning of this article, the original landing place was in the outer harbor about three-fourths of a mile inside the present breakwater, on what is now a United States reservation. About the time of the Civil War the landing place was shifted to the northern shore of Wilmington Bay. With the completion of the two jetties and the deepening of the harbor, the Southern Pacific Railroad was extended across Wilmington Bay, in 1882, to the San Pedro Bluffs near Timms Point, which place then became the principal shipping center on account of the much deeper water at the latter place.

The commerce and draft of boats so largely increased that a project to get 15 feet was adopted in 1881. This naturally contemplated building a west jetty from Timms Point, and the extension of the east jetty beyond Deadmans Island to the 3-fathom curve. The scour of the tide was also to be aided by dredging the hardest portion. This work was completed in 1893 and was successful in giving a depth of 16 feet at mean low water across the bar.

Then, as always, it was found that commerce and the draft of vessels using the port were pushing its capacity, and plans for further development were made. These included dredging to get a depth of 18 feet on the bar and for deepening and straightening and, in some places, widening the channel inside the entrance.

This was promptly authorized, but the work was delayed until the question of where a breakwater for a harbor "for commerce and of refuge" should be constructed in the vicinity of Los Angeles.

The commerce of the harbor, which was 50,000 tons in 1871 when improvements were begun, had risen in 1888 to 450,000 tons. With the comparatively small ships of that time, this tonnage meant the arrival and departure from the harbor of several hundred ships each year.

There being no anchorage completely protected from storms, a ship was occasionally wrecked by being driven ashore from its moorings. These wrecks caused a strong demand for a breakwater to form a harbor of refuge. With this demand for a breakwater, first considered about 1880, there began a fight between an energetic city and a powerful railroad for the control of Los Angeles Harbor. The city wanted a harbor in the best place for one and

above all, a harbor open to all men alike. The railroad wanted only a second-rate harbor where its control would be absolute. It took twenty years to settle this one question of a breakwater on the side of right and justice—twenty years filled with examples of the corrupt use of money, of political intrigue, and of magnificent courage of men battling for a square deal against seemingly overwhelming odds. The battle is still in progress, but now shifted to the question of whether the public shall absolutely own and control the harbor, with every prospect of the complete early triumph of the public. No further reference will be made to this

Fig. 2. Solid rock portion of east jetty, 2,000 feet in length, connecting the wooden "double work" with Deadmans Island.

fight except when necessary to explain engineering details, though the story is one filled with dramatic incidents worthy of a Shakespeare.

Between 1886 and 1896, four different projects for a breakwater in San Pedro Bay were submitted to Congress. (See Plate V.) The first, by the Engineer officer in charge, was for a breakwater about 7,000 feet long, formed with two arms separated by a gap of 1,000 feet. The inner end of the breakwater was to be on the 3-fathom curve just off Point Fermin. The inner arm was to extend seaward to about the 9-fathom curve and the outer arm was to follow a straight line approximately on that curve at a total estimated cost of $4,000,000. The project of 1890 by a board of Engineer officers was for a similar breakwater, but with the

inner arm beginning at high-water line and extending only to the 6-fathom curve, followed by a gap about 1,300 feet in width. The outer arm was to follow nearly the same line as in the first project, but to be 4,000 feet in length. The estimated cost was $4,500,000.

The project of 1892 by a second board of Engineer officers was for a continuous breakwater 8,500 feet in length, built on a curve until it became tangent to the line of the outer arm of the 1890 project and then to follow that line to the end. The estimated cost of this project was $2,885,000.

The fourth project (1896) was submitted by a board of engineers composed of an officer of the Navy, a member of the Coast and Geodetic Survey and three civilian engineers. Their project was very similar to the project of 1892, except that the breakwater began at the 4-fathom curve and was located in slightly shallower water, the depth along the outer curve being 8 fathoms. The total length was to be 8,500 feet and to cost $2,900,000. The cost was fixed by the act of Congress providing for the board, and the conclusions of the board were to be final.

Due to advantageous contracts the work was done cheaply enough to allow for lengthening the breakwater 750 feet at the outer end, making its total length 9,250 feet. The breakwater up to low water is a rubble-stone mound. The stones below low water weigh from 100 pounds to 15 or 20 tons, two-thirds averaging over 1,000 pounds each. As designed, the width of the sub-structure at low water was 38 feet, but this has been increased to about 48 feet by a berm on the harbor side.

The superstructure begins at low water and ends 14 feet above, the two walls being laid in courses with rectangular blocks of granite. On the ocean side there are four courses, each 3½ feet thick, no stone weighing less than 16,000 pounds. The harbor side is built up of seven courses, each 2 feet thick with no stone weighing less than 6,000 pounds, while the interior is filled solidly with all sizes of stones. The width between the outer edges of the bottom courses is 38 feet, and at the top 20 feet. The courses above the bottom on the ocean side are each set in 3 feet 4 inches, and those on the harbor side 1 foot 4 inches.

The original design of the breakwater was faulty in one very important particular. The slopes on the harbor side and below a plane 12 feet below low water on the ocean side were assumed at 1 on 1.3, while the slope on the ocean side from low water to the plane 12 feet below low water was 1 on 3. No berm was provided

for either side, the width of the rubble mound at low water being 38 feet, or the same as the width of the bottom of the superstructure. However, even before any considerable construction work was done, it was deemed advisable to provide a berm not less than 4 feet wide on the harbor side to protect the inner wall from being undermined by the masses of water that experience, with breakwaters on the Great Lakes and elsewhere, had shown would be dashed over during very severe storms.

No berm was placed on the ocean side until a section of the superstructure had been in place long enough to show that some, at least, of the stones in the bottom course on the ocean side would be undermined by the back wash of the waves unless a berm were provided. While it was not thought that this would in any way affect the stability of the breakwater, yet if the symmetry of the work was to be maintained, a berm was necessary. Accordingly, for all extensions of the breakwater thereafter, as well as for those portions of the substructure already completed, sufficient stone was deposited on the ocean side to make a berm approximately 5 feet in width.

In addition, several thousand tons were deposited along the bottom course on the ocean side of the superstructure, where the latter was already partially or wholly completed. This work was expensive and difficult to execute; it being necessary, as a rule, to roll stone off cars by crowbars or similar means, and to make matters worse, these falling stones occasionally displaced or even broke up some of the wall stone above the bottom course.

In the construction of the breakwater a double track standard gauge railway trestle was built along the line of the work, the bents being about 16 feet apart, center to center, and each containing four perpendicular and two batter piles. (See illustration, Fig. 3.)

Two travelling cranes converted from steam shovels were used to unload the heaviest stone—both superstructure and substructure. These cranes were prevented from tipping over, when the boom was nearly at right angles to the car, by steel guys fastened to the track on which the loaded cars were resting while the crane was on the other track alongside. In addition, very heavy brackets carrying two rollers were fitted to each side of the front of the crane. The rollers rested on a movable 4-foot platform of 12-inch timbers supported on other 12-inch timbers placed on top of the caps of each bent between tracks.

Obviously, without the double track system these cranes could handle stone but a very few feet from each side of the car, and along the single track left where the superstructure was practically complete the crane was of no great use in unloading stone that was wanted 22 feet outside of the center of the track. This was the case with the stone used to form a berm along the bottom course on the ocean side where the superstructure was already complete.

The specifications for the work required the rock in the substructure to be dumped and brought up to a little above low water at least six months before any superstructure stone was placed, to allow the substructure to adjust itself to any settlement that might occur. This, of course, left a very irregular foundation on which to place the wall stone, and in order to get the bottom stones approximately level it was very often necessary to pull out large stones from the substructure. The resulting holes were filled with smaller stones up to the level of low water. It was these small stones particularly that the back wash of the waves on the ocean side displaced, and in a few cases allowed the bottom stones on the ocean side to slide downwards and outwards partly out of place. While this occurred in perhaps a half dozen places, it is not believed any considerable displacement of wall stones will occur, even where no outer berm was originally provided and where the amount of stone later deposited for this purpose was insufficient to form a proper berm.

In building the superstructure much of the piling was sawed off just above low water and the railroad tracks blocked up by these pieces of piling, which were further shortened from time to time as the superstructure gained in height. This was for the purpose of making the density of the superstructure as great as possible by replacing with granite the space occupied by the piling.

The breakwater, as planned in 1896, was completed in 1910, and the work of extending it westward to the Point Fermin shore is now (1911) in progress with a rubble mound.

It shelters from storm waves an area of 370 acres having from 30 to 52 feet of water at low tide, and over 200 acres more having 20 to 30 feet at low tide. This anchorage is easy of access at all times. The entrance is 4,000 feet wide, with a depth shoaling gradually from 50 feet at low tide at the breakwater to 30 feet at the inner line. No matter how severe the storm, so long as a ship can maneuver at sea the anchorage can be entered. Likewise, crippled vessels travelling under their own power or being towed

can reach quiet water and safety when drawing as much as 48 feet of water. These are advantages that very few harbors enjoy.

A project to deepen the inner harbor was adopted and money appropriated in 1896, but its expenditure for that purpose was made to depend upon the report of the special breakwater board of 1896 referred to above. If the board reported *in favor* of San Pedro Bay, the expenditure was *not* authorized.

In January, 1900, Capt. James J. Meyler, Corps of Engineers, deceased, submitted in a most able, far-sighted report, a project for

Fig. 3. Crane, handling squared stone for the sea wall of the breakwater. These stones are granite, $3\frac{1}{2}$ feet thick, and weigh not less than 16,000 pounds.

deepening the entrance to the inner harbor to 20 feet over a width of 400 feet and to deepen a mile of the inner harbor, beginning at the wharf nearest the entrance, to a depth of 24 feet for its full width. This width varies from a minimum of 500 feet 1 mile north of the entrance to 1,600 feet at the turning basin at the inner end, over 2 miles from the entrance.

Captain Meyler discussed at length the practicability of deepening the entire bay of Wilmington and the possibility of its being needed in the not distant future. His partial project up to and

including the turning basin, 2 miles inside the entrance, was adopted, money appropriated, and work begun in 1903.

The writer assumed charge of the Los Angeles District February 10, 1906. At that date there remained to complete the breakwater the delivery and placing of 500,000 tons of stone, and to complete the partial project of 1900 the dredging of 3,000,000 yards of material.

The work was so well organized, both as to personnel and equipment, as to require comparatively little attention from the main office.

The real task before the office was to plan for the future development of the outer harbor, as well as the 1,350 acres of bay above the turning basin in the inner harbor. Harbor lines had been adopted for the outer harbor, but they were not satisfactory. Indeed, steps had been taken before the office came under the writer's charge to change the lines of the outer harbor.

As for the inner harbor above the turning basin, all was uncertainty and chaos. A board had been appointed and harbor lines recommended, in 1905, along the general lines of Captain Meyler's report. (See Plate III.) Attention is here called to the fact that the Southern Pacific Railroad crosses the inner harbor from north to south so as to divide it into two nearly equal parts, known as the east and west basins. The harbor line board of 1905 proposed to form, east of the Southern Pacific Railroad, a huge basin from 1,500 to 2,400 feet in width and some 3,200 feet in length, where ships might anchor as well as maneuver. The west basin was to be developed in the same general way, but with a comparatively small basin. Everywhere both pierhead and bulkhead lines were shown. In order that open-work piers might be built of sufficient length to accommodate future ships, these lines were placed 600 feet apart where there was lowland sufficient to warrant it and at lesser distances in other places.

But just here all progress was stopped by a ruling of the Engineer Department that the War Department had no authority to establish harbor lines above the turning basin.

In December, 1906, an application was made by interested parties for a permit to reclaim the bay east of the Southern Pacific Railroad by the construction of two channels, each 1,000 feet wide, with a peninsula between them from 1,100 to 2,000 feet in width and some 9,000 feet long. (See Plate IV.)

This peninsula plan, as it was called, was faulty in four import-

ant particulars: First, the complete control of the water frontage of the peninsula by private parties would have been very easy, because it could be approached in only one direction, and that across nearly three-quarters of a mile of private land; second, it provided for no turning basin other than the channels themselves, which were only about 1,000 feet in width; third, it made intercourse by water between the different parts of the harbor difficult, as, for instance, if one desired to go from the east end of the north channel to the east end of the south channel or to the Long Beach Harbor, he would have to make a trip of more than 3

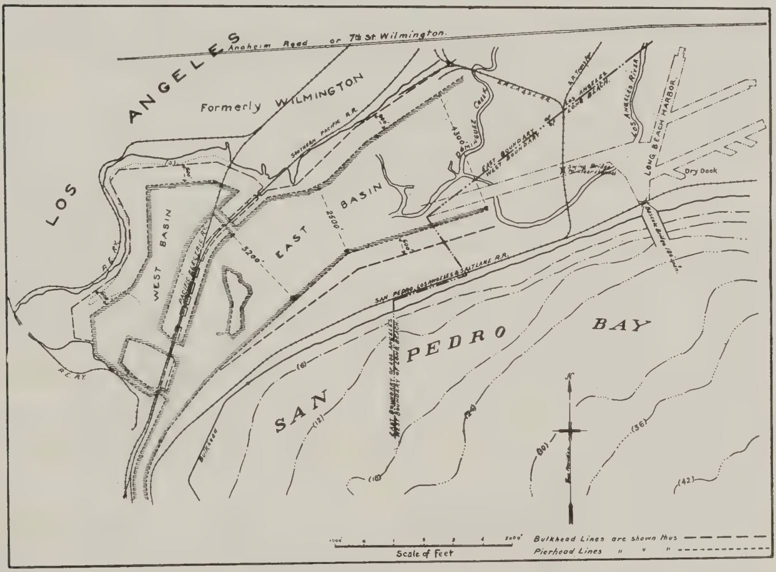

Plate III. Hachured lines are pierhead lines proposed for the inner harbor in 1905.

miles to gain a distance of only ½ mile; and fourth, the natural approach to the entire East Basin from the north through the town of Wilmington would have been rendered valueless except for the frontage along the north side of the north channel. Even this frontage would have been greatly reduced in value, due to the small amount of land for wharves, yards, etc., between it and the Southern Pacific Railroad, which would have made control by the latter comparatively easy. After two and one-half months work, the office declined to recommend the peninsula plan, and submitted to the Department a plan for the development of the

entire inner harbor, both east and west of the railroad. (See Plate V.)

This plan provided for two basins, as in the 1905 plan, but reduced the water areas to that actually required for maneuvering and passing purposes, and made pierhead and bulkhead lines only 40 feet part. A minor Y-shaped basin was made in front of the old landing at Wilmington, and areas to be reclaimed so arranged that slips could be dredged into the land when more water frontage than that along the harbor lines was needed. A plan similar to this had previously been worked out for the outer harbor, the main change from old lines there being to push the bulkhead line out to 18 feet of water, while at the same time providing for two channels 400 and 600 feet in width, respectively, and extending shorewards distances of 2,600 and 4,000 feet.

These plans for the inner and outer harbors changed the whole conception of the harbor when considered as a whole. Up to this time, 1906, the general idea had been that the breakwater was primarily to form a harbor of refuge during storms and that vessels having business in the harbor would pass directly into the inner basin upon arrival at the port, and if no dockage space were available they would anchor there. It was on this theory that the 1905 board proposed the great inner east basin.

This was reversing the natural order of things by causing vessels going to anchorage to pass through the dockage areas first, and relegated the breakwater's usefulness to giving refuge to an exceedingly small number of distressed ships, and to shelter such piers as might be built in the outer harbor.

The new conception is to make the outer harbor primarily an anchorage basin while still being available as a port of refuge, and for such harbor facilities as may eventually be constructed there. For this purpose the outer harbor was given as great development as practicable, keeping in mind that ample land for dockage, warehouse, and terminal purposes must be available only a few feet above high water level.

This condition did not exist naturally, as the bluff, from the breakwater around to Timms Point, averages 50 feet in height. By pushing the bulkhead line out into 18 feet of water at low tide and providing for two channels with a total length of 6,600 feet, the plans provide for 16,000 feet of frontage along bulkhead lines and 350 acres of reclaimed land in this outer harbor. The frontage can be increased somewhat if required by the construction

of slips dredged into the area to be reclaimed nearest the breakwater.

The development of water frontage by the construction of openwork piers is vastly inferior to development by slips, and the harbor that must depend on piers—whether necessary, or built as a result of wrong ideas—will be at a great disadvantage when compared with those harbors developed with slips.

As before stated, a gap of 1,900 feet was left between the shore and the beginning of the original breakwater, but, in 1908, upon

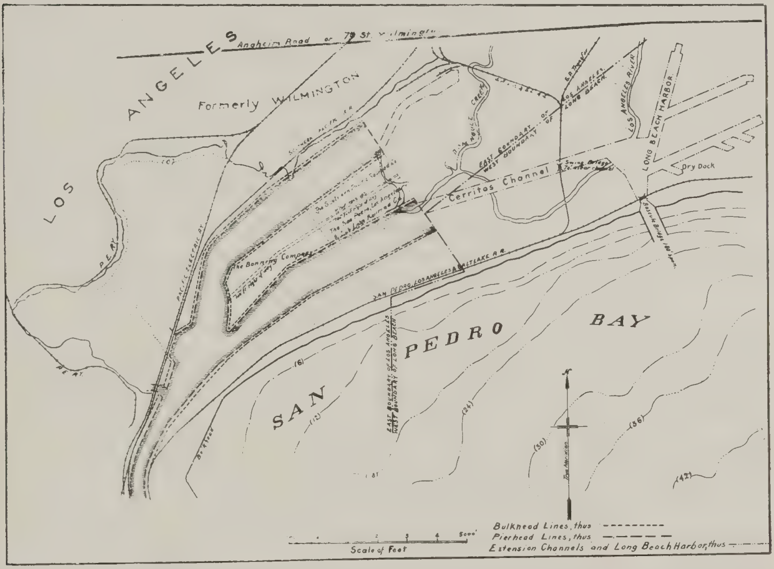

Plate IV. Hachured lines show Peninsula Plan, proposed by private interests for the east basin of the Inner Harbor in 1906. Note that no provision was made for the west basin.

the recommendation of the Los Angeles office, a project was adopted for closing the gap. The original idea of the gap was that it would be used by ships both in calm weather and during storms, and allow currents to carry silt and sewage out through the opening.

The depth of water at the outer end of the gap is 24 feet at mean low tide, shoaling to 18 feet at a point about 600 feet shoreward. From the latter point to the shore the bottom rises gradually to the high-water line, and then precipitously to a height of about 40 feet above mean low water. Point Fermin and a shoal making out therefrom protect the gap between the breakwater and the shore

from westerly and southwesterly seas, but for southeasterly to southerly storms there is practically no protection for any portion of the 1,900 feet of opening, and the waves that run through the deeper part of the gap during such storms are large enough to make very rough water over a large area inside.

According to the plan now adopted, the bulkhead line extends out to within 620 feet of the west end of the breakwater. This would leave a gap of 620 feet between the breakwater and the bulkhead on the south side of the land to be reclaimed. The waves coming through this portion of the gap cause the greatest disturbance inside the breakwater, and would cause great inconvenience to wharves and ships along the outer edge of the proposed fills.

It had been argued that in the future this gap might be used by small boats and even vessels drawing between 20 and 25 feet of water, not only during fair weather but particularly in stormy weather, in order to gain as quickly as possible the shelter of the breakwater. Inasmuch as a vessel intending to make the harbor through this gap would have to traverse kelp and to pass near and for some distance parallel to a rocky reef, this contention is not worthy of serious consideration.

As to silt from the inner harbor, no considerable amount could be carried out through this opening by the currents. On the other hand, during storms the waves undoubtedly stir up along the shore outside of the breakwater a considerable amount of sand, carry it through the gap, and deposit it in the quiet water inside. Currents passing out through the gap, even though they might be of sufficient velocity in the gap to carry a considerable amount of silt, can not have sufficient velocity in the much wider cross section inside to stir up or even transport anything but the very finest material. It would appear, therefore, that the opening would be a detriment rather than an aid, so far as shoaling inside the breakwater is concerned. The work of closing the shore gap in the breakwater is now well under way.

Plate V shows the outer and inner harbors of San Pedro as definitely planned, according to official harbor lines and constructions undertaken or authorized by the General Government, together with suggested extensions including the Long Beach Harbor.

The outer harbor has been sufficiently described, but it should be noted that the areas marked A and C are owned by the city of Los Angeles by special grant of the State legislature, and the area marked B, while being extensively improved by a private corpora-

tion, is in litigation between the city and the corporation. It will also be noted that the United States still retains its military reservation of 40 acres and the land to be reclaimed in front of it with 1,400 feet of frontage. This reclaimable land was granted to the United States by the legislature of California, in 1897, in an act giving to the United States exclusive jurisdiction for 300 yards out beyond the low-water line over all waters adjacent to United States reservations for military, naval, and other public purposes.

The entrance to the inner harbor begins at Deadmans Island (another United States reservation) where the channel is nearly 700 feet in width. This width remains nearly constant for 2,000 feet, at which point it begins to widen gradually and is 1,000 feet wide 3,000 feet inside the island. From there it narrows rapidly to 500 feet where the channel has a 3-degree curve for a distance of 3,000 feet. This width of 500 feet, coming as it does on a 3-degree curve less than a mile from the entrance, is the most serious objection to any part of the harbor. It can, however, be very easily increased to 650 feet by moving the wharves on the east or convex side of the curve back 150 feet.

Along this portion of the inner harbor the east jetty is 200 feet back of the present pierhead lines, and even if the channel is widened 150 feet there will still remain a strip of frontage 50 feet wide controlled by the public through franchise provisions regardless of who may control the area east of the jetty. Probably the best way to bring about this widening with the least cost and with the least disturbance to business would be to make all franchises covering this area for short variable periods, all to terminate at one time, say, in 1920 or 1925, at which date the widening to 650 feet would go into effect. In this connection it may be stated that one of the strongest arguments for an ocean entrance to the Long Beach Harbor (see Plate V) was that it would in the future relieve the congestion through the Deadmans Island entrance and provide against the possible contingency of this latter entrance being temporarily closed by a wreck.

The inner end of the section 500 feet wide is 7,500 feet inside the island, and from that point the channel widens again gradually to the turning basin which, though originally made 1,600 feet in diameter, has been so extended by the 1908 harbor lines as to allow a circle 1,700 feet in diameter to be inscribed in a quite irregular water area, the irregularity being caused by three main channels

opening into it from the north and east. The upper end of this basin is 12,000 feet from Deadmans Island.

From the turning basin a 600-foot channel leads to the west basin. This channel is being spanned by a double-track single-leaf trunnion type drawbridge, having a clear opening of 180 feet.

Two thousand feet south of this main channel to the west basin is a second 200-foot channel spanned by a fixed bridge. This bridge should have a clear head room of about 14 feet at low tide, as the channel is designed especially for the use of launches, row boats, lighters, etc.

The west basin has an area of 630 acres below high tide lines, 210 acres of which will be in channels and basins, leaving 420 acres to be reclaimed. The frontage available along Government bulkhead lines is 29,000 feet. This may be increased economically about 5,000 feet by digging slips.

As mentioned later, caution must be observed in digging slips not to make too many, thereby decreasing below the economically efficient point the storage, warehouse, and yard areas.

At the northwest corner of the west basin the harbor lines extend to the high-water line. This was done in order that if conditions ever warrant it and anybody desires to do so, a channel can be dug a half mile north into Bixby Slough and more frontage developed therein. About 3 miles north of this slough is another very much larger one, called Nigger Slough, which has already been more or less exploited as a harbor possibility. Personally, the writer believes that the cost of improving these sloughs is too great to make them a commercial possibility until the other vast areas, mentioned later, are developed, and that is a very long look into the future.

On the east side of the Southern Pacific Railroad a channel 750 feet in width connects the turning basin with the main east basin, the future center of the entire harbor. A 500-foot channel leads from this main basin to a Y-turning basin along the old Wilmington water front, and another channel east of Mormon Island extends nearly 4,000 feet in the same general direction, so as to make of Mormon Island a peninsula about 100 acres in extent. From the northeast corner of the east basin a channel 3,000 feet in length has already been dug to very extensive lumber yards.

A gap 700 feet in width was left in the southeast corner of the

east basin harbor lines in anticipation of a channel to be dredged to Long Beach Harbor, 7,000 feet farther eastward.

In return for the privilege of filling in the old tortuous bed of Cerritos Slough, the Salt Lake Railroad Company has deeded to

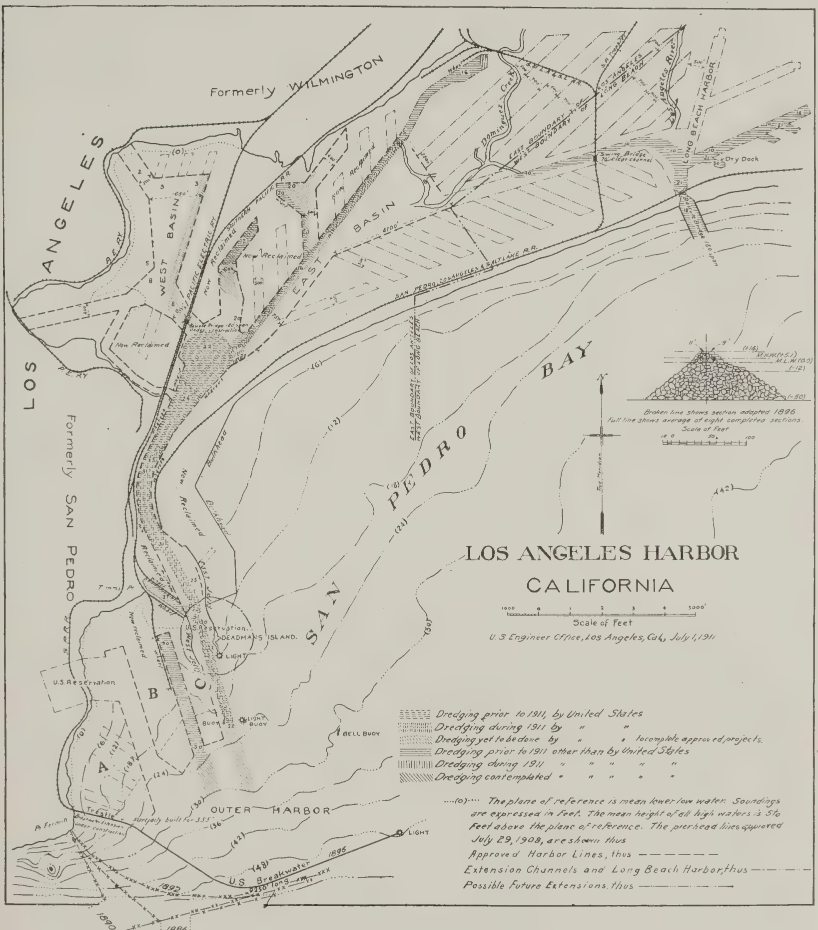

Plate V. Showing present approved harbor lines and possible extensions of Los Angeles Harbor, and the same for Long Beach Harbor and the area between the two harbors. Also the breakwater as actually constructed, together with the location of earlier projects. (See page, 13.)

the United States Government a strip of land 400 feet wide connecting the Long Beach Harbor with the east basin. This channel has been partly dredged, so that light-draft launches can pass through at all times. In the opinion of the writer, this channel

should be 700 feet wide, and it was only when it appeared that it was necessary to accept a 400-foot strip or postpone indefinitely the getting of a straight channel that the acceptance of the Salt Lake Railroad Company's offer was recommended. As a matter of fact, the 400-foot channel will do very well as a connecting channel between the Long Beach Harbor, with its ocean entrance, and the east basin, and that is all the public is vitally interested in in that locality, since the holdings on either side are private.

The railroad company is the one that must have a wider channel if it ever makes the best use of the 1,500 acres, more or less, of magnificent harbor property the company owns on both sides of this channel and on the south side of the east basin.

In the east basin there are 35,000 linear feet of bulkhead lines and of the total area of 660 acres between the south line of the main channel and the high-water line of the bay in front of Wilmington; almost exactly half (330 acres) is in channels and basins and half (330 acres) land to be reclaimed.

About seven-eighths of Wilmington Bay was patented by the State of California to individuals as swamp and overflowed land and tide land under a perversion of United States and California State laws intended to apply to reclamation for agricultural purposes only. In 1908, suit was instituted by the State to recover these lands on the ground that the patents were invalid, and on January 3, 1911, a decision was rendered in the Superior Court of Los Angeles County declaring the patents void. The decision was so sweeping as to include long time grants, and if, as seems assured, this decision is confirmed by the Supreme Court of the State, the city of Los Angeles will control about 800 acres of reclaimable land inside the inner harbor and about 56,000 feet of frontage along bulkhead lines.

In addition, the city now controls about 215 acres and 7,000 feet of frontage in the outer harbor, and 30 acres and 4,000 feet of frontage in the inner harbor.

To recapitulate briefly: The breakwater is 11,150 feet in length, and shelters 370 acres of water from 30 to 50 feet deep at low tide and over 200 acres more with depths of from 20 to 30 feet outside of established harbor lines. Much of this 200 acres will in time be deepened in getting material to make fills in the outer harbor and dredging channels to deep water.

The outer harbor proper has two main channels 2,600 and 4,000 feet, respectively, in length, and 350 acres of reclaimable land with

16,000 feet of bulkhead lines. This frontage may be increased economically by slips, so as to give 7,000 feet more frontage.

Between Deadmans Island and the turning basin in the inner harbor there are 18,000 feet of bulkhead lines and 500 acres of land, most of which is now reclaimed. The frontage there may be increased a few thousand feet by slips. Above the turning basin, in the east and west basins combined, there are 750 acres of reclaimed land and 52,000 feet of frontage exclusive of the Salt Lake Railroad Company's land on the south and east. This frontage may be increased economically by slips to the extent of 13,000 feet.

The above is along present approved harbor lines, and while the total—including the slips suggested by the writer and the 12,000 feet along the Salt Lake Railroad Company's property—amounts to 132,000 feet, or 25 miles, it does not represent half the frontage that can be developed if the future shall show that more is needed.

Bounded by the bluffs of San Pedro on the west, the Anaheim Road and the city of Wilmington on the north, the city of Long Beach on the east, and San Pedro Bay on the south, there are some 8 square miles (about 5,000 acres) of swamp, tide, and submerged lands capable of being practically improved as part of the inner harbor. Before this is all developed it is evident that the present anchorage area will be too small.

However, due to the fact that the ocean bottom becomes flatter as you go east from Point Fermin, the breakwater can be extended to inclose any amount of anchorage that may be desired. Indeed, a 25,000-foot extension, should that much ever be required, could be made on almost the same line as the outer arm of the present breakwater, and while keeping in depths averaging barely 48 feet would inclose 10 square miles of water, half of which would average more than 36 feet.

As to the cost of an extension, experience with the present breakwater indicates that the expensive squared wall stones can be omitted from the superstructure, thereby materially reducing the cost per foot.

Not only can the breakwater be greatly extended, but if fifty or one hundred years hence a long extension becomes necessary, the harbor frontage itself can be increased at least 17 miles by the construction of nine slips between Deadmans Island and the entrance to the Long Beach Harbor. The slips beginning at the present 18-foot curve could be made in lengths varying from 6,500 to

2,100 feet, with a tongue of land 1,000 feet in width between each two slips.

It would seem advisable, whenever any considerable extension of a breakwater is made, to leave a gap 2,000 feet in width between the present breakwater and the beginning of the extension.

Considering again the inner harbor, the land owned by the Salt Lake Railroad Company between the Long Beach Harbor and the east basin can probably best be developed by slips opening into the east basin and the Cerritos Channel between it and the Long Beach Harbor, giving a frontage of 12½ miles. On Plate V the channels are shown 500 feet wide on the north side of Cerritos Channel, where they are about 1 mile in length and 300 feet wide on the south side where the lengths are 2,000 feet.

In addition to the foregoing, there is the Long Beach Harbor, owned by private corporations, with the exception of 2,000 feet purchased by the city of Long Beach.

This harbor has an ocean entrance 500 feet wide, partially protected by rubble jetties beginning at the high-water line. Just inside the high water line, the Salt Lake Railroad crosses the channel by a Scherzer rolling lift bridge with a clear span of 180 feet.

The Long Beach Harbor is being constructed in 800 acres of land purchased by a corporation for that purpose. A great deal of work has already been done and wharves and a shipbuilding plant, with a dry dock, are already in operation there. This harbor, along the lines shown on the map, has about 3½ miles of frontage, and this can be increased economically 1½ miles or, possibly, a little more by the construction of other slips.

The city of Long Beach and the private parties doing this work deserve much credit for standing by their convictions under circumstances that cause most people to fail.

It may be noted that the percentage of frontage suggested for different areas varies to some extent. This is in a large measure due to the fact that basins are taken from certain areas alone, while they serve other areas equally well. Furthermore, where slips are directly in front of the mainland the percentage of frontage to land between slips may be slightly increased.

As regards widths of channels, the writer believes 300 feet about the minimum safe width where the largest ships may be expected. In slips under 1,500 feet the width may be narrowed to 250 feet, but cramped channels like cramped wharfage space spells delay, danger, and increased cost of handling freight. In general, a

400-foot width is suggested for slip lengths of from 2,000 to 4,000 feet and still greater widths for lengths beyond 4,000 feet.

There is ample room in this harbor to allow for liberal channels and liberal land areas between, and short-sighted indeed will be the man who builds or advocates building slips on other than liberal lines.

The question of maintenance of depths is always a very important one when considering the future of a harbor. In this matter Los Angeles Harbor is exceedingly fortunate. Indeed, it is hard to conceive of an ocean harbor that will cost less to maintain.

Fig. 4. Portion of breakwater near shore end, showing harbor wall completed and single line of track. San Pedro Hill is just off the right edge and Point Fermin is just off the left edge of illustration.

The two causes of the deterioration of a harbor are silt carried down by rivers flowing into the harbor and sand piled up at the entrance by cross currents and wave action.

Ordinarily, the Los Angeles River is the only one whose waters reach the harbor during the rainy season. During the greater part of the year the river goes entirely dry before reaching the sea, due to irrigation and the great quantities of water used in the city of Los Angeles. It is noted, however, that during the winter of 1910-1911 the San Gabriel River, which ordinarily flows into Alamitos Bay, about 10 miles east of Los Angeles Harbor, broke from its regular channel into one known as New River at a point about 20

miles from the harbor and, following the New River, united with the Los Angeles River at a point about 5 miles north of the harbor. This was one of the worst floods known in many years and carried into the Los Angeles Harbor possibly 350,000 cubic yards of material. Efforts are now being made by railroad companies and agricultural interests in the vicinity of the break to make such improvements in the bank of the river as will confine it in the future to its regular channel, emptying into Alamitos Bay.

The Government has been asked to aid in this as a measure of protection to the harbor, and steps are being taken in that direction. Unquestionably this improvement will be made, but even if the San Gabriel River should regularly flow into the Los Angeles Harbor, the cost for dredging would still be comparatively small, as the records for nearly fifty years show only five serious floods. These occurred in 1867, 1873, 1884, 1891, and 1911. The Los Angeles River itself carries down some material in smaller floods at lesser intervals, but the amount is so small as to be scarcely noticeable, except just where the river first enters the deep water of the harbor.

The writer would not have the Government appropriate money to divert the San Gabriel River until after sufficient funds are available to deepen the inner harbor, up to the turning basin, to 35 feet and the channels around Mormon Island to 30 feet.

As regards wave action, it was noted before the breakwater was begun that the entrance to the inner harbor kept its depth remarkably well. With the completion of the breakwater, all wave action has been stopped and no deterioration whatever will come from that source in the future.

The present condition of the Los Angeles Harbor is well shown on the map accompanying Captain Leeds report on the harbor for the year ending June 30, 1911. (See Plate V.) By this it will be observed that the 400-foot channel in the outer harbor has been dredged to a depth of 30 feet at low tide.

From Deadmans Island northward the channel is deepened to 30 feet to the middle of the curved portion, where the width is only 500 feet. The dredging to 30 feet above Deadmans Island is being carried on by the Government dredge. Notwithstanding the dredge was out of commission for two months during the fiscal year 1911, a total of 764,000 yards was removed at a little less than 9 cents per yard. Seaward of this area work is being carried on by contract with a clam-shell dredge and a depth of 30 feet has been

obtained for a width of 200 feet, nearly the entire distance from the 30-foot contour in the outer harbor up to the point where the Government dredge began work, and a contract has been let to deepen the entrance to 30 feet for a width of 400 feet. The remainder of the channel up to and including the turning basin has already been dredged to 25 feet at low tide by the Government dredge. This latter dredging cost, including repairs and deterioration of plant, an average of about 8.6 cents per yard.

Above the turning basin, channels along the northern side of the east basin have been dredged to depths of from 16 to 20 feet for widths of from 100 to 200 feet. Also, a portion of the Y-turning basin in front of Wilmington has been dredged to 20 feet, while the city of Los Angeles has let a contract to dredge 1,200,000 yards in the Y-basin and the channel to the west of Mormon Island, where some dredging has already been done.

As before stated, work is now in progress constructing a drawbridge to the west basin, under orders from the War Department to have it completed by December 31, 1911, and a contract has been let by the United States to dredge a 200-foot channel through the draw into and along the east side of the *west* basin.

The entire work as far as the turning basin will be completed to a depth of 30 feet, and the channels above the turning basin to 20 feet before December 31, 1912.

The commerce passing through Los Angeles Harbor during the fiscal year ending June 30, 1911, was over 1,700,000 tons, an increase of 20 per cent over that for 1910.

So much for the past and present, but what of the future? Just now the outlook could not well be brighter, and yet if the harbor is made what it should and can be made, continued vigilance, sense, and energy must be used to prevent private interests from so directing harbor improvements as to benefit them at the expense of the public. The early opposition to locating the breakwater at San Pedro, as well as the later opposition to the adoption of proper harbor lines in the inner harbor, was based on the desire of private interests to absolutely control all harbor frontage serving Los Angeles and the surrounding country. The winning of these two fights did not destroy the opposition, though it has forced them to adopt new tactics.

Now public ownership and operation of harbors, like public ownership and operation of any other public utility, is the people's greatest protection against private greed. With the consolidation

of the harbor district and Los Angeles, in 1909, it became evident to every one that the public was determined to own and operate portions, at least, of the harbor. If public ownership of the harbor could not be prevented, the only sensible course for the opposing private interests was to delay that ownership and make it as expensive and unproductive as possible.

Thus we see certain interests, both before and after consolidation, trying to force or cajole the city of Los Angeles into improving on an elaborate scale the most expensive, distant, and inaccessible part of the harbor—that is, the portion of the outer harbor between the breakwater and the Government reservation.

The inner harbor is from 1 to 3 miles nearer the city of Los Angeles than is the outer harbor. It is easily approachable in nearly every direction, in addition to being much cheaper to develop and operate. That, then, is the place for all public harbor improvements until years hence when the inner harbor has about reached the limit of its capacity. By inner harbor, in this connection, is meant all the channels and the adjacent land on both sides from Deadmans Island inward.

This idea of the relative value of the inner and outer harbors is not new and, as the following extracts will show, was held by able, disinterested engineers familiar with the harbor from ten to twenty years ago, before local rivalry had been thought of.

In 1894, Col. W. H. H. Benyaurd, Corps of Engineers, U. S. Army, made the following statement in his report:

> Beyond the upper end of the wharves the basin expands. Ultimately the extension of commerce and the interests of the harbor will require the dredging of this portion.
>
> It appears desirable to dredge the inner channel as far as the head of the wharves; and ultimately to enlarge the interior basins as the demands of commerce will warrant.

Later, in 1897, a board of which Admiral J. G. Walker was chairman, was appointed by the President to locate a deep water harbor at Port Los Angeles (Santa Monica), or at San Pedro, Cal. This board reported, in part, as follows:

> While the extension of the jetties and the deepening of the inner harbor may not be a necessary part of the immediate construction required for the deep water harbor provided for by law, they certainly are accessories which would within a reasonable time and at a reasonable cost enable this harbor to meet every possible requirement of commerce in the most complete and satisfactory manner.

In comparing Santa Monica and San Pedro, the board further states:

At each place a good deep water anchorage in 6 to 8 fathoms of water can be obtained. At either place the necessary accessories for the convenient transfer of business from shipping to land can be constructed. At Port Los Angeles these accessories must be in the form of piers built out into the sea, and of such other facilities as may be afforded on made land behind a bulkhead wall. At San Pedro corresponding provisions can be best furnished by the improvement and deepening of the inner harbor.

The board also stated that:

The whole of Wilmington Lagoon is available whenever the needs of commerce demand it. Within this inner harbor vessels could lie perfectly still in all weathers without even the reduced sea which would exist within the protected area behind the breakwater. * * *

Prior to the completion of these improvements of the inner harbor, it is reasonable to assume that one suitably designed timber pier, located a safe distance on either side of the jetty entrance and practically carried out to the 5-fathom line, would accommodate those vessels whose draft would prevent their entrance to the inner harbor, * * * but it should be regarded, if constructed, as a temporary feature of the deep harbor development, and not as a permanent part of it.

The board also states:

The series of examinations made under the direction of this Board also show that any further improvement that may be needed can readily be made, and that the possibilities for the further development of the inner harbor are equal to any demand upon it which the future can be expected to make.

In the broad consideration of this question, therefore, it must be assumed that the improvement of the channel and interior harbor at San Pedro will be continued. (Senate Document No. 18, 55th Congress, 1st session.)

Capt. James J. Meyler, Corps of Engineers, U. S. Army, in his report dated January 6, 1900, submitting project for the improvement of the inner harbor, stated:

It is a land locked harbor, its waters are calm, and vessels within could lie perfectly still in all weathers without even the reduced sea which would exist within the protected area of the outer harbor.

The inner harbor will be much easier of access by the railroads than will the outer harbor; the grades will be less and the distances shorter; the piers that will be necessary to reach deep water need not be so extensive, and the cost of maintaining them will be far less than in the outer harbor; and accommodation for the maintenance

and repair of vessels can be erected in the inner harbor much more conveniently at hand. In fact, the inner harbor will be much more than an accessory to the outer harbor, and with both completed the latter will serve essentially as an anchorage ground and a harbor of refuge, and the former as a harbor of commerce for the direct and convenient connection of ship and rail. * * * The people of this section realize fully that in comparative usefulness and relative importance to commerce the latter (inner harbor) affords many times the advantages that will result from the former; and that the outer harbor can only assume its greatest value when the inner harbor has been fully improved.

As regards the actual development of the inner harbor in the future, it will be difficult to make any vital mistake, and the writer would urge only four things: 1. Begin deepening to 35 feet just as soon as the 30-foot depth is carried up to and including the turning basin, which will be about December 31, 1912. At the same time, deepen to 30 feet and for nearly their full widths the channels on both sides of Mormon Island leading to Wilmington. Also extend channels 200 feet wide and 20 feet deep wherever needed.

2. Leave the most inaccessible part of the outer harbor alone until the inner harbor can not accommodate the commerce of the port.

3. Widen to 650 feet all portions of the channel below the turning basin that have not that width now, by moving the piers back on the east side of the harbor the necessary 150 feet or less, as required.

4. Extend public ownership and operation to all parts of the harbor as fast as the city's finances will allow it.

Summing up, there are 132,000 feet or 25 miles of frontage available in the inner harbor with nearly 1,250 acres of reclaimed or reclaimable land, not including the deeded property of the Salt Lake Railroad. In the outer harbor there are available 350 acres of reclaimable land and 4 miles of frontage, with an ample and secure anchorage basin. In addition, there is sufficient low land owned by the Salt Lake Railroad, or included in the Long Beach Harbor, to afford 17½ miles additional frontage whenever needed.

This is an enormous amount of frontage, and most people will be inclined to question its ever being required. It must be remembered, however, as stated in the beginning of this paper, that there are only four commercial centers on the Pacific Coast of the United States, and considering the whole frontage that can be developed no other harbor can equal Los Angeles in such vast development on

lines that will reduce the cost of handling to the absolute minimum. Throughout the whole plan, as frequently referred to, ample land is provided for railroads, storage yards, permanent and temporary warehouses and sheds on the peninsulas between each two slips.

If any one doubts the tremendous value of this condition, let him study the present New York and San Francisco water front problems, where storage is reduced to narrow and expensive wharves. Then study Antwerp, Hamburg, and other progressive European harbors, all of which are making extensions by means of wide slips with still wider tongues of land between. There the saving of a few cents on each ton of business means the continued growth of the harbor, and the loss of a very few cents on each ton the decline of the harbor—at least relatively—to other nearby harbors.

European harbor builders realize keenly that narrow wharves, congestion, rehandling and increased cost are inseparable, and in all extensions are avoiding them by every means in their power.

If the foregoing plans or others on broad lines are kept in view, and the public shall own, control, and operate the harbor, who can say that in seventy-five or one hundred years all of it will not be in use? A great start has been made. Los Angeles has already voted $3,000,000 in bonds for building a belt line railroad, wharves, storage yards, etc., and bound herself to spend $7,000,000 more in public harbor improvements with the ultimate aim of owning and operating the whole harbor.

And Los Angeles is right, for unquestionably it is only through public ownership and operation that the harbor can become great. Private ownership and control of any harbor means no improvement until congestion forces it, and then only on the lines that will give most profits to the corporation. Public ownership, on the other hand, means the construction of ample facilities as fast or even a little faster than actually needed, thereby attracting shipping while charges are just enough to pay for extensions by long-time bonds, and for maintenance and operation.

In conclusion, public harbors are to-day as much of a necessity as public highways. Indeed, they are an integral part of our highways, and for the people to go on spending millions on river and harbor improvements without providing public terminals and public harbors is as absurd as if they were to build beautiful macadam roads and then allow private parties to erect toll-gates at every milestone.

Treatment of the Foundation for the Power House and Dam, Hales Bar, Tennessee River

BY

Mr. C. H. TISDALE
Junior Engineer

The construction of a dam across the Tennessee River at Hales Bar to provide power for generating electricity has been carried on by a private company for several years under a contract with the United States. At one end of the dam it is proposed to build the power house in which to install the machinery, and at the other end a lock is provided to pass vessels from one level to the other. The dam will be 41 feet high above the low water surface, and from 50 to 65 feet high above the rock foundation. A high head is thus to be created. All structures are to be founded on solid rock and are to be of concrete.

All foundations at this work are in a formation known as ''Chickamauga limestone.'' The appellation ''Chickamauga'' is local, and is a distinction in name only. The formation known as limestone is one of the most widely extended geological formations known. Probably its most familiar characteristic is the number of seams, both horizontal and vertical, that occur all through it and that cut it into distinct blocks, more or less. On this work these seams vary in thickness from that of a knife blade or less, in which case the seam is called ''tight,'' to 12 inches or more. They are usually filled with clay, shale, or rotten rock, more susceptible, of course, to mechanical disintegration than the rock proper. The effect of ages of scour upon a river bed of this character may be imagined. As existing in the foundation under discussion, these seams are most usually filled with a smooth, grainless, yellow clay or mud saturated with water. It is evidently a deposit from the river water. The side walls have been hollowed out and wrought into grotesque chambers that stir the mind to speculation. Some have been encountered 20 or 30 feet below the surface, containing quantities of sand and gravel of varying sizes. This proves an

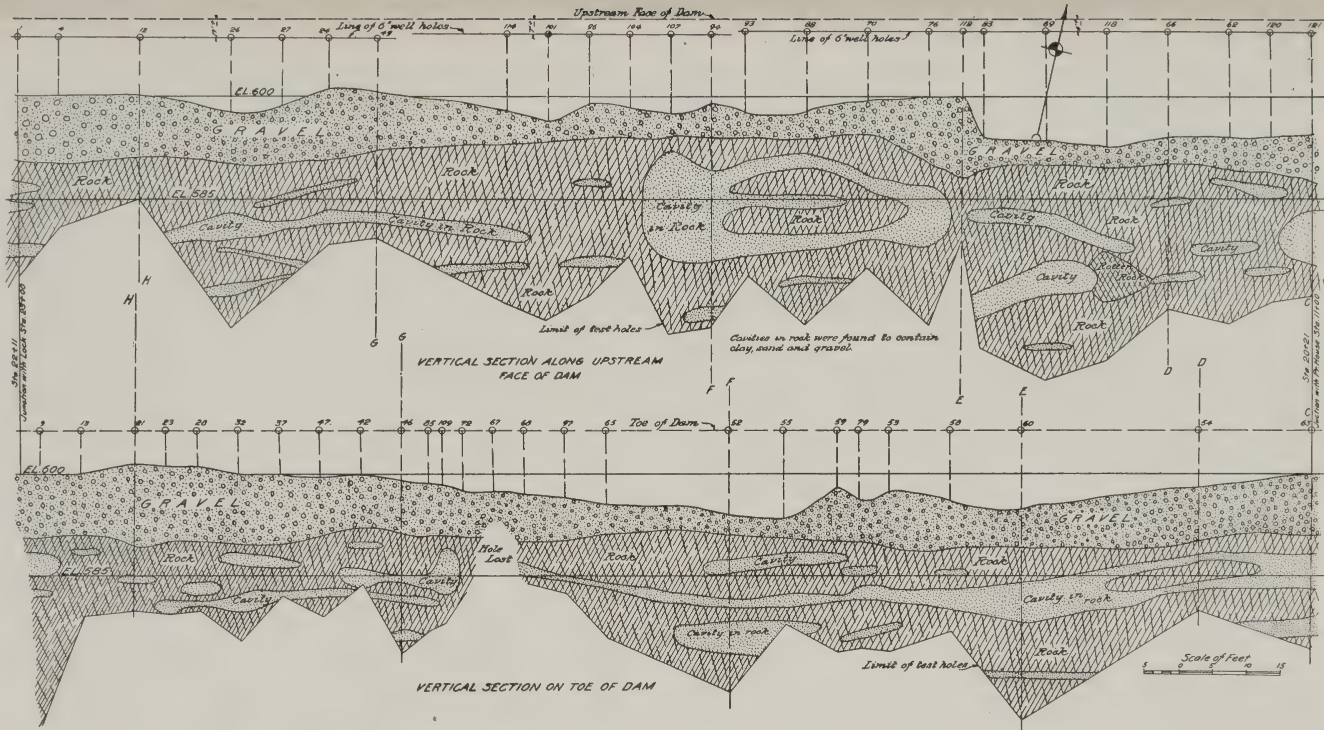

Fig. 1. Tennessee River, Hales Bar Dam. Sections along face and toe, showing probable condition of foundation. Prepared under the direction of Maj. Wm. W. Harts, Corps of Engineers, U. S. A.; C. H. Tisdale, Junior Engineer. Note: Outlines of cavities are obtained by connecting corresponding surfaces as developed by 6-inch well drills. Treatment of foundation consists in removing all top shell less than 2 feet in thickness and filling exposed cavities with concrete. Cavities more than 3 feet below surface to be reached by drill holes, and soft material washed out with water under pressure and refilled immediately by forcing in grout.

existing connection with the river above, a condition of the foundation that must certainly be reckoned with to guarantee the integrity of the upper pool, especially at low water stages. As a commentary upon the extent of the leakage into the coffered sections of the dam through the foundation (the dam is coffered into four sections of about 300-foot length each), it may be stated that two of three sections now coffered could not be drained, even to the gravel, before the leaks were treated. The general elevation of the rock on the dam site is about 9 feet beneath low water level, with from 5 to 8 feet of gravel overlying it. The depth of footing required is from 3 to 6 feet, dependent upon the character of the rock, and beneath this it is the endeavor to obtain about 10 feet of foundation through which practically all leakage is stopped. It is known that beneath this level there are yet other openings and passages in the rock that perhaps will require sealing, but their treatment will constitute an operation separate from, and probably additional to, that necessary to secure stability alone.

The meaning of the word "treating" in this discussion is synonymous with grouting. If the contents of the seams and caves in the foundation could be solidified or removed to make way for grout, a substantial result would be achieved. In view of the depths at which these conditions obtain, excavation in the open was unfeasible. Results could have been obtained with the use of caissons and compressed air, but this would have been an extremely costly and very slow method.

The logical thing to do, then, was to treat subsurface conditions from the surface through well holes and percussion drill holes sunk from 15 to 40 feet into the rock. This was done. The procedure was to mount well drills on floats, from which the holes were sunk. The operation was not unlike driving a well on land. The water was usually pumped down in the cofferdam low enough to locate the main leaks. These locations then became especial points of attack. A hole once down and piped, about an 80-pound air pressure was applied. This usually caused the water to seeth and boil over an area 50 feet square, more or less. The points showing the greatest disturbance were then attacked and other holes sunk.

In drilling, an outer 8-inch casing was placed, through which the hole was started. As drilling proceeded, this casing was driven down till it rested upon rock. After drilling 3 feet into rock, another casing (6 inches) was placed inside the 8-inch casing and tightly driven into the rock. After the hole was finished, this

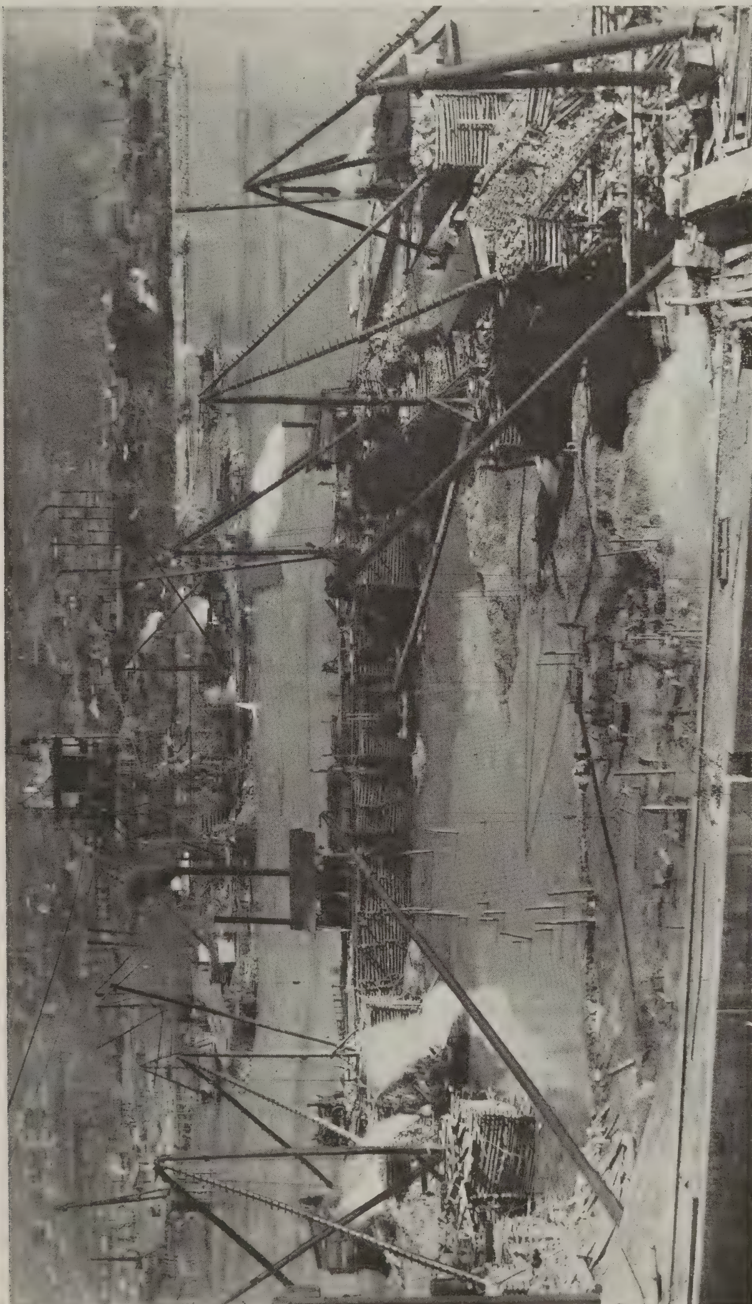

Fig. 2. General view of plant and site at Hales Bar, Tennessee River, from lock wall.

inner casing was extended to an elevation 2 feet above the water level outside the pit and covered with an ordinary blind cap, or plugged, or else reduced to receive the 2-inch connection with the grouting machine.

After a sufficient number of holes were down, and all were piped and capped, the air was applied to one and the others were uncapped one or two at a time. In some instances a pump was connected with the holes and water forced through. In this way different combinations of air and water jets were used to loosen the contents of seams and cavities and clear them out. The next step was to flood the pit and begin grouting. Usually, during this operation a head of water of a foot or more was maintained in the section over the outside river level. The idea was to make the grout flow with the water through the subterranean channels into the river. An ordinary grouting machine was used, and the usual charge was 280 pounds of cement (three bags) and about 10 gallons of water.

This machine consists of a cylindrical tank of 40 gallons capacity, through which passes a shaft fitted with paddles for mixing the charge. The mixing is done by a motor at the rear of the machine, which is operated by compressed air. After measuring in the charge of water and cement, the trap at the top is closed and the batch mixed. From 5 to 15 pounds of air is then admitted to the cylinder, which forces the grout through the flexible connection into the pipe being grouted. The holes were always filled before stopping. After all holes were filled and allowed to set two or three days, the pit could generally be drained. The gravel overlying the rock was then removed and percussion drilling was begun. Holes 16 feet deep, spaced about 4 feet, were drilled around the area over which concrete was to be laid. Dependent upon the degree of leakage through these holes and the condition of the seams and openings beneath, grouting was again in order. The effect of grouting operations conducted in the dry through the small drill holes was nearly always successful. Such a procedure, however, required a fairly tight top surface in order to hold the grout underneath. Progress under these conditions was faster and the effects could be more accurately gauged than where the pit was flooded. On the other hand, flooding the pit was necessary where the rock surface was too much cut up to hold the grout beneath it. In two instances it was found impossible to grout a very shallow, though water bearing, horizontal seam some 4 feet below the footing in the dam. These seams were known to be there because of the leakage

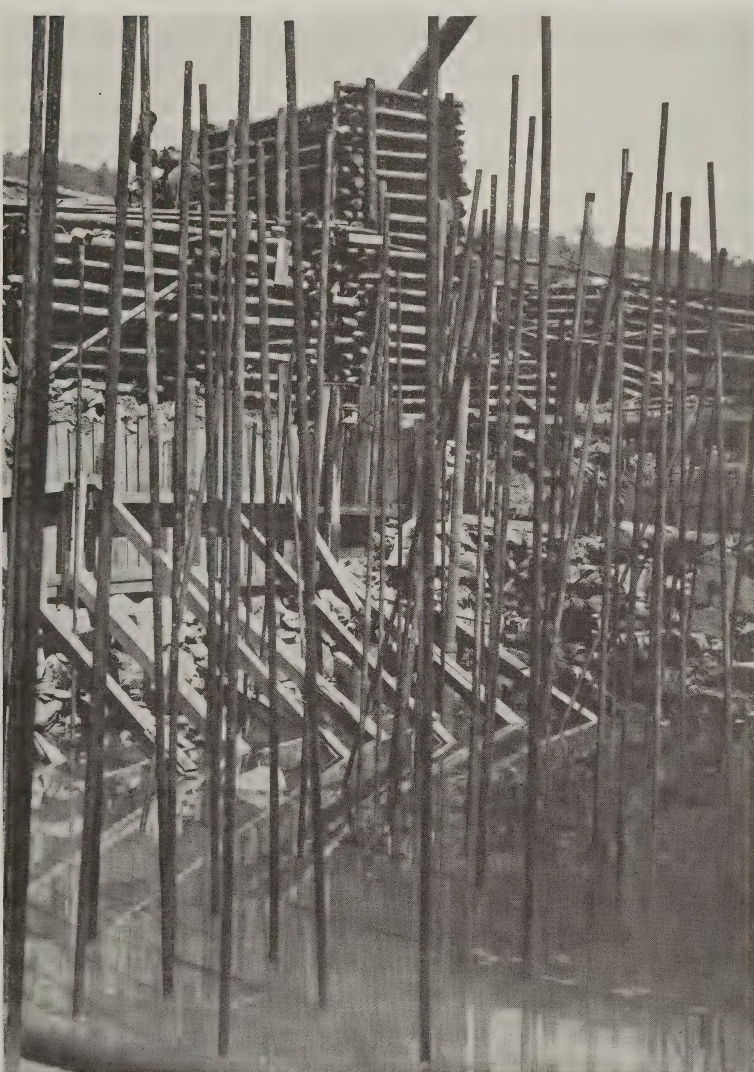

Fig. 3. Second dam section at Hales Bar, piped and ready to "flood" before grouting; the pipes extend a few feet above the water surface outside of the pit. This method was used where the bottom was not "tight" enough to hold the grout. The holes were drilled in the dry.

through the test holes. They were so shallow that the percussion drills would not show their presence. In these areas anchor rods were grouted over the affected surface, extending 4 feet below the seam and 4 feet into the concrete. The surfaces requiring this treatment were not large. The precaution of using these anchor bars or rods along the face of all dam footings has been adopted. The rods are of 1 inch section, cold-twisted structural steel of an ultimate strength of 60,000 pounds. They are spaced about 3 feet apart and extend 10 feet into the footing, along a line not over 8 feet from the upstream surface. The holes are filled with a thick grout and the rods dropped in. If the holes leak so that the grout will not be retained, they are extended below the leaky seam 3 or 4 feet, provided it be within 10 feet of the footing, then filled with grout and the rod dropped in. The grout below the seam stays. This can not be done in case of a heavy discharge from the hole without using a pipe, through which the grout is introduced at a point below the seam. Occasionally, in leaky holes or seams in the dam footing, it has been expedient to concrete the pipe in place, the water being conducted through a pipe extension outside of the concrete and grouted later.

In the power house a more serious condition was developed in the foundation than any previously encountered in the dam. Here the excavation was carried about 35 feet into the rock. The deeper the excavation was carried the heavier the leakage through the bottom became, until some 20 feet of rock excavation had been made. At this level a blast uncovered a large water-worn vertical crevice—that we found later extended down some 20 feet farther into rock—through which gushed about 15,000 gallons of water per minute. This leak flowed for about one year before being grouted. At the expiration of that time three Keystone well drilling machines (size No. 3) were put on floats and some 30 holes were put in the first dam section next the power house pit and two were sunk in the power house pit itself. These holes averaged about 35 feet in rock. About one-third of these holes were dry, no openings or crevices of consequence being struck. The remainder, however, including the two in the power house pit, encountered seams and openings, at elevations from 15 to 25 feet below the rock surface, some of which were 18 inches thick. A number of these holes were "gushers," or would have been had the tops of the casings not been above the river level. The main leak in the power house pit before mentioned was simply "piped"—that is, a 4-inch pipe was thrust some 5 feet

down into the crevice and carried some 10 feet above the river level. It was grouted by gravity alone, no machine being used. Machines were used on all the holes drilled. The water level in the pits was kept from 6 inches to 3 feet above river level during the grouting. This head was, of course, too great, and resulted in the loss of a large percentage of the cement used. Fifteen thousand bags of cement were used in this grouting, 2,000 of which were used in the large leak mentioned. Both pits were dried thereafter without trouble. The cavity, which was the source of the heavy leak in the power house pit, was later excavated for about 15 feet—that is,

Fig. 4. The leak in the power house pit that held the contractors at bay for eight months or more. The discharge was estimated at about 15,000 gallons per minute. It was finally piped as shown and grouted by gravity alone, no machine being used, and a perfect job secured. The rock at this leak was 15 feet above grade.

to where it passed into the foundation below grade, which at this point is about 40 feet below low water in the river. It had been scoured clean of all mud by the water and filled full of hard grout that hugged the sides of the passage so closely that the joint could hardly be detected. The grout seemed as hard as the rock itself.

After this, while the excavation was proceeding in the dry, seven more holes were sunk 20 feet below grade in the power house pit.

In each hole a mud seam was encountered about 18 inches thick at an elevation a few feet below grade, that gushed a 6-inch stream about 10 feet in the air. The pit was flooded as before, and 1,700 bags of cement were used in grouting. A few days later in blasting near grade, this seam was again reached and the pit was filled with very muddy water in twelve hours. A hole was put down at this point, grouted, and the pit finally drained again. Then the percussion drills were brought in to test the bottom preparatory to concreting. They revealed the presence of a horizontal layer of mud or saturated clay lying from 2 to 5 feet below grade, extending indefinitely up and down the river and about 70 feet

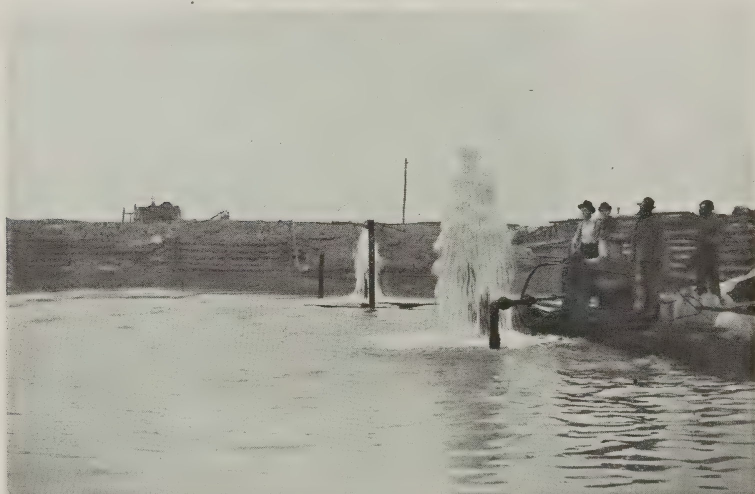

Fig. 5. The effect of compressed air at 50-pound pressure on one 40-foot 5-inch hole in the dam foundation. The pit is shown flooded and ready for grouting.

wide across. It varied from 6 inches to 5 feet thick, and was under water pressure everywhere. The programme adopted by the contractors was to drill holes over the affected area with percussion drills, all of which were piped and capped. Through these holes alternate jets of air and water were forced. Then grouting through these pipes followed. It was then discovered that the grout was finding its way into the pit through a large opening at one end of the affected area. In order to continue grouting in the dry, the whole area was covered with concrete in which steel rods were placed. This concrete was the floor of the power house. Then be-

gan a steady campaign of drilling and cleansing by air and water jets as before described, and grouting. The idea was to clean out the mud and fill with grout. This was continued for nearly four months, and over 400 holes were drilled and grouted over an area about 50 by 70 feet. About 3,000 barrels of cement were used in this grouting. Probably about two-thirds of the mud in the cavity was replaced with grout. It lay along the bottom and the grout was on top. The solidity of the grout was such that a half-inch rod could not be thrust into it by hand. The drill, though, would drive a bit through it almost as easily as through the mud below. All water flow was practically cut off. The foundation was accepted in this shape.

The following conclusions have been formed as to how grouting operations should be conducted under conditions such as obtain here: (1) The more thoroughly the seams and cavities are cleansed, the more certain will be the effect of the grout; (2) the head of water in the pit over the outside water should be not over 1 or 2 inches during the grouting; (3) the air at the machine should be used sparingly. Ordinarily, a 5-pound pressure is plenty, and it should not be allowed to enter the pipe; (4) less water should be used in the grout; (5) grouting in the dry is, wherever possible, a more certain operation than grouting with a flooded pit; (6) grouting in the dry, especially when the drilling is done by percussion drills, is much cheaper, more convenient and rapid than any other way.

Vertical Lift Bridges

BY

Maj. W. D. Connor*
Corps of Engineers; Member American Society of Civil Engineers

It is difficult to realize that a little over sixty years ago it was a much debated question as to whether or not the railroads of the country would be permitted to bridge the navigable streams crossed by them. Prior to the construction of the bridge across the Mississippi River, at Rock Island, Ill., it was not known whether the courts would permit the railroads to bridge the Mississippi River and other navigable waters, or whether the rivers would have to be crossed by means of ferries.

The decision of the court in the case of the bridge at Rock Island, as well as the numerous court decisions concerning the bridging of navigable waters since that date, can be summarized in the statement that there is no objection to bridging navigable waters, provided the bridges are so constructed as to permit traffic through or under them with reasonable ease and safety.

Where conditions are favorable, the simplest method of satisfying both the demands of navigation and the requirements of continuous rail traffic is to construct the bridge of such height that all boats using the stream can pass under the bridge at high water. In many localities the construction of a bridge of that height is out of the question, and it becomes necessary to resort to tunnels under the waterway or to bridges with one or more movable spans.

The first and most common type of movable span is the ordinary swing bridge with two equal spans. The width of the clear openings of the draw spans is fixed by the War Department, as one of its duties connected with the control of the navigable waters of the United States. These clear widths are based on the probable needs of navigation during the ordinary life of the type of bridge proposed. In a bridge of several spans, the swing truss is considerably heavier than the fixed trusses of equal length, due to the fact that

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it is a composite structure acting as a cantilever to support itself when open, and as an ordinary truss to support its own weight and the weight of the live load when the bridge is closed. The extra weight thus made necessary, together with the weight of maneuvering machinery and other accessories, requires the pivot pier to be very much larger and therefore more costly than the other piers in the bridge. In addition, a long pier of cribwork or pile construction must be built the full width of the pier and length of the truss, parallel to the current, in order to protect both pier and shipping and partially support the swing truss when open.

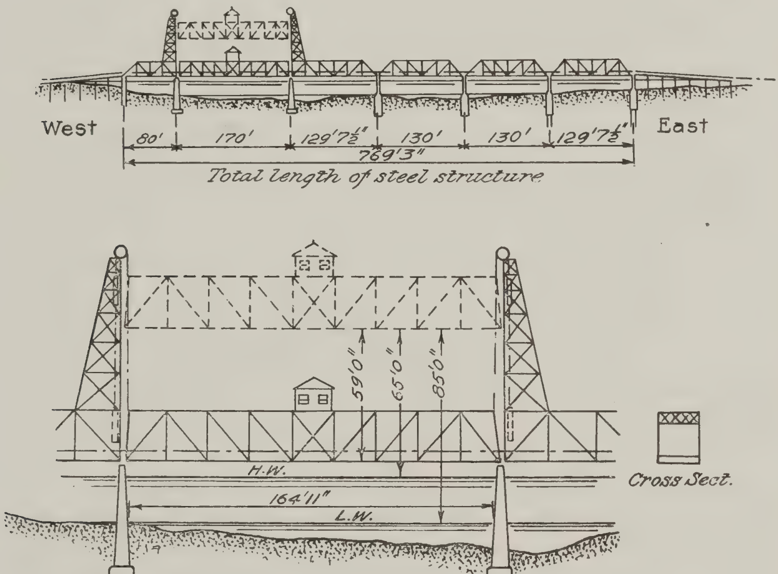

Fig. 1. Vertical Lift Bridge over the Sacramento River at Tehama, Cal. Ordinary highway traffic is on a paved roadway 20 feet in the clear. The channel span is raised to accommodate river traffic and is fully balanced by concrete counterweights. It is operated by alternating electric current. The span is raised to its full height or lowered in 90 seconds. Masonry piers support the main spans and the end spans are supported by steel cylinders filled with concrete. The approaches are timber trestles and earth fill. The structure when completed will cost about $175,000. Waddell & Harrington, Consulting Engineers.

The most serious objection to the swing bridge, and one which in many localities has made it in later years absolutely prohibitive, is the space taken up by the center pier, which not only narrows the clear span available but is an obstruction both to the current and to the passage of boats. These objections led to the develop-

ment, nearly twenty years ago, of bascule bridges of the Scherzer rolling lift type and the trunnion type. These types meet the objections based on the space taken up by the old center pier, but are not free from the objections based on heavy pier construction, space in which the counterweight may roll, as in the Scherzer type, or in which in revolves, as in the trunnion type, and other objections such as patent rights, reversing strains, etc.

The vertical lift type described below gives promise of meeting many of these difficulties, and, if the present success with them continues, should become a very common type.

Figure 1 shows the essential points of this class of bridge. It consists of a series of ordinary trusses. On the ends of the trusses adjacent to the opening, towers are constructed provided with sheaves at the top over which pass cables fastened to the ends of the lift truss at one end, and to a counterweight at the other. The height of these towers depends upon the clear headway required by the War Department, the depth of the truss, and the height of the lower chord in its normal position above high water. In Fig. 1 the trusses are the ordinary Howe trusses with the addition of vertical end posts at the ends adjacent to the opening. These form part of the tower on the two fixed trusses, and are the carriers on the lift truss. The counterweights at each end are approximately equal to the half weight of the truss. They are made of concrete and, when the lift truss is up, form a gate closing the adjacent spans. The truss is raised and lowered by means of auxiliary cables attached to the top and bottom of the towers and passing over drums in the operating house. By turning these drums in one direction the truss is lifted, and in the other direction the truss is lowered. Since the counterweights and the truss are of equal weight, practically the only work done by the operating machinery is to overcome the friction of the cables and sheaves. Each pier adjacent to the opening thus carries the entire weight of the truss plus one-half the weight of the adjacent truss, including the weight of the tower; but, inasmuch as the lift truss is a much simpler affair than other types of draw spans, the weight on the pier is considerably less than would be necessary in case either other type were used.

Figure 2 shows an elaboration of this design. The upper deck of the bridge is used for a highway, and provides a double track street railway between the trusses, an 11-foot roadway and a 6-foot sidewalk on the outside of each of the trusses; while the lower deck of the bridge carries a double-track railway. The ordinary traffic

on the Willamette River is such that it can readily pass beneath the level of the highway deck, and only masted vessels require greater headway. In order to provide for the ordinary requirements of navigation without disturbing the entire traffic on the bridge, the part of the lift span which carries the railway is simply suspended from the corresponding panel points of the upper truss which carries the highway traffic, and the lower deck can be drawn up close to the lower chord of the highway deck. In case masted vessels desire to go through the bridge, this lower deck is drawn up to the highway deck and the entire lift span is then raised to the position shown in dotted lines on Fig. 2. The suspenders

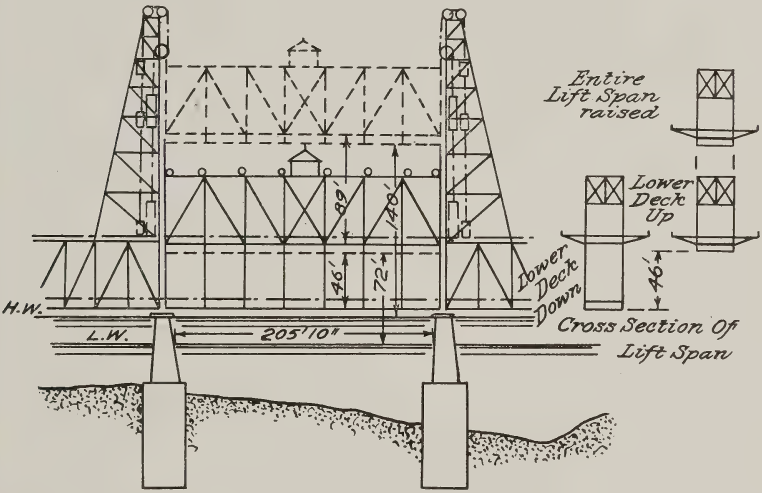

Fig. 2. Vertical Lift Bridge of the Oregon Railroad and Navigation Company, over the Willamette River at Portland, Oreg. The lower deck provides for a double-track railway in a roadway 32 feet 8 inches in the clear. The upper deck provides for a double-track street railway in a roadway 29 feet 3 inches in the clear between trusses, vehicular traffic on two cantilevered roadways 11 feet in the clear and pedestrian traffic on two sidewalks 6 feet in the clear, making 72 feet 9 inches between outside hand rails. The lower deck in the channel span is raised to accommodate river traffic; and to permit the passage of high-masted sailing vessels, the lower deck telescopes with the upper deck and then both are raised to a clear height of 140 feet above high water. The total weight of the lifting span and deck is 4,300,000 pounds. A 12-inch gas main is carried across on the lift span, remaining in continuous service during the movement of the span. The span is operated by direct electric current from street railway mains. The lower lifting deck is raised to its full height or lowered in 30 seconds, and both decks are then raised to their full height or lowered in 60 seconds. The lifting span and lifting deck are fully and independently counterweighted. Waddell & Harrington, Consulting Engineers.

of the lower deck are drawn up inside of the corresponding members of the truss. The lower deck can be raised or lowered in thirty seconds, and the entire lift truss can be raised to the position shown in dotted lines in Fig. 2 in one and one-half minutes.

The cost of a swing span or bascule at times seems excessive where only a small highway bridge is to be constructed and where the demands of navigation require an opening to be provided; and these vertical lift bridges are so simple and comparatively inexpensive that they may form a very satisfactory alternative to the swing bridges and bascules.

On rivers with shifting channels like the Missouri and the Arkansas they offer decided advantages. At Fort Smith and Rob Roy, Ark., it was necessary to spend considerable money on controlling works to maintain the channel through the draw span; and at Omaha, Nebr., Sioux City, Iowa, and Rob Roy, Ark., an extra draw span had to be constructed owing to a change in the location of the channel. With vertical lift bridges the towers and the counterweights can be moved, and at a small cost a new lift span can be provided over the new position of the channel.

Dredging at Muscle Shoals Canal with Ladder Dredge

BY

Mr. A. D. Edwards
Junior Engineer

The Muscle Shoals Canal is divided into two parts, locally designated as the upper and lower divisions, which are separated by 8 miles of open river. The upper division consists of two locks connected by a mile of canal, an upper pool 2½ miles long, and a dredged channel below the lower lock. The lower division is composed of 14½ miles of canal and nine locks. Fifteen streams, varying in size, empty directly into the canal, though none of them are very large, yet at every freshet they bring down a certain amount of sediment, and bars are constantly forming in the channel opposite their mouths. At the entrance to both divisions of the canal a large amount of silt also accumulates at every high water, and constant dredging is therefore required to keep it cleaned.

A Bucyrus dredge of the elevator type is employed on the canal for this purpose, having the following general dimensions:

Hull.—Length, 80 feet; width in center, 38 feet; width at each end, 13 feet; depth of hull, 6 feet. The sides are circular, being struck with a 68-foot radius so as to give the above dimensions. Draft when working, 42 inches.

Buckets and links.—A chain of twenty-four buckets and twenty-four links is mounted on a ladder frame 48 feet long, equipped with truss rods and fittings, rollers with shafts and bearings, top and bottom tumblers, device for holding bucket chain, and hoisting tackle for regulating the depth of cut. This chain of buckets works over the forward end of the boat, and slopes back at an angle of 45 degrees until it reaches an elevation of about 22 feet above the deck, where the material is discharged into a hopper. This

chain of twenty-four buckets, each having a capacity of 5 cubic feet, makes one complete revolution in one and one-fourth minutes, discharging 4.44 cubic yards per revolution, which gives the dredge a capacity of 213 cubic yards per hour, or 2,130 cubic yards per day of ten hours.

Discharge pipe.—From the hopper, which is located 15 feet above the center of the boat, a discharge pipe 26 inches in diameter and 80 feet long, suspended by a set of tackle attached to an A-frame, conducts the material that is dumped into the hopper to the place of deposit, which is usually beyond the tow-path. When the material is thick and heavy, a stream from a 6-inch pump is turned into the pipe to keep it flushed out.

Driving engines.—The dredge is equipped with a 10 by 14 inch double cylinder engine making 140 revolutions per minute, developing 40 horsepower.

Swinging engines.—These engines are double cylinder, 8-inch diameter and 8-inch stroke.

Boiler.—The boiler is of marine type, 40 horsepower, 60 inches in diameter, 17 feet long, and carries a pressure of 90 pounds.

Pump.—For flushing the discharge pipe a Gordon Duplex steam pump is used, having 12-inch steam cylinders, 10-inch plungers, 16-inch stroke. Capacity, 326 gallons per minute.

This dredge, when in operation, revolves about a center spud, which is 40 feet from the point of the buckets, thus enabling a cut 80 feet wide to be made. The depth of the cut varies from 3½ to 10 feet below the surface of the water.

This dredge has an advantage over other types, as it cleans the entire width of the canal as it moves forward, and deposits the material outside of the canal bank, where it does not have to be handled again. The canal is cleaned with a single cut, with the exception of a few places where two cuts, and sometimes three, have to be made before the material is finally deposited outside the canal bank. This method is a little slow, but it is the best way to handle it, as it would not be practicable to load the material in scows and tow them outside of the canal. Above Lock A (upper division) three cuts have to be made, and between Locks 1 and 2 (lower division) three cuts are necessary. Another point where some difficulty is experienced in operating the dredge is above

Lock 1, where the banks are too high for the discharge pipe to reach over them. To dredge this part of the canal the river has to be caught at a stage that will allow the discharge pipe to clear the bank.

The crew necessary to operate the dredge consists of one dredge runner, one engineman, one fireman, one spudman, and two linemen.

The hull of this dredge was built at Chattanooga, Tenn., by contract, in 1891. The machinery was placed on hull and floated to the canal, where the cabin was built and machinery installed. The

Fig. 1. Ladder dredge at work in Muscle Shoals Canal.

total cost of dredge was approximately $20,000. The hull was rebuilt at the canal by hired labor in 1902 and 1903, at a cost of $10,000, being put back in commission in October, 1903. The dredge has been operated almost continuously since it was rebuilt. A new hull will have to be built within the next year or two, also a complete set of new buckets and links will be required. The machinery is in good condition and will outlast another hull.

The following statement gives the number of cubic yards dredged with cost of labor, material, and field repairs since the dredge was put in commission.

Year.	Cubic yards dredged.	Unit cost.	Total cost.
1892	27,210		
1893	38,964		
1894	42,800		
1895	13,235		
1896	5,513		
	127,722	$0.036+	$4,725.54
1897	61,550	.039+	2,440.37
1898	62,097	.041+	2,586.39
1899	39,375	.036+	1,455.44
1900	59,200	.038+	2,288.25
1901	18,093	.144+	2,615.84
1902	55,764	.031+	1,753.96
1903	38,123	.028+	1,102.08
1904	100,012	.030	3,000.00
1905	105,490	.031+	3,291.10
1906	146,968	.030	4,409.04
1907	111,337	.023	2,560.75
1908	59,372	.056	3,324.83
1909	117,777	.046+	5,507.43
1910	195,982	.036+	7,065.98
*1911	87,731	.036+	3,174.75
	1,386,593	$0.036+	$51,302.04

*To January 1, 1911, of fiscal year ending June 30, 1911.

Total cost of labor, material, and field repairs	$51,302.04
Cost of rebuilding hull in 1902 and 1903	10,000.00
Deterioration of plant	10,000.00
Total cost of dredging since 1892	$71,302.04

This gives for the unit cost of dredging 1,386,593 cubic yards of material, $0.05142 per cubic yard.

Report on Physical Characteristics of European Seaports*

BY

Mr. CHAS. W. STANIFORD
Member American Society of Civil Engineers
Chief Engineer, Department of Docks and Ferries
New York City

HON. WILLIAM J. GAYNOR,
Mayor, City of New York.

SIR: During the years 1908, 1909, and 1910, I visited and studied the plans and methods of the principal seaports of Europe. During the latter year, Mr. Charles W. Staniford, Chief Engineer of the Department of Docks and Ferries, also visited these ports, and his studies, more recent and more complete than mine, are embodied in the following report.

It is desirable that this information be presented for public consideration to guide the efforts which New York and other American ports are making to modernize harbor conditions. Heretofore the policy of New York, and of most American seaports, has been to build separate docks without relation to each other either as regards construction or use. The policy which the city has now entered upon contemplates the organization of the port as a whole, with a view to correlating the several parts and planning each district for its best natural use.

New York Harbor is so spacious and so much room has been found for manufacturing and commercial enterprises about its shores that until recently no great inconvenience has resulted, either from crowding or bad organization, and the necessity for planning harbor facilities has not been as apparent as elsewhere. Even now there is no pronounced congestion except in one section, the west side of Manhattan, where competition for space between the railroads, steamships, the local commerce of the river and Sound, and the prospective commerce of the Erie Canal has resulted in serious congestion and disorder, very detrimental to the commerce of the city. Like conditions will soon develop elsewhere unless a comprehensive port plan and policy shall be adopted.

In the several reports which I have made to your Honor during the past year, I have outlined the main features of a comprehensive plan and policy for the port, with especial reference to the west

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side of Manhattan, South Brooklyn, Staten Island, and Jamaica Bay. The essential feature in each of these reports is a railroad parallel to the waterfront connecting the docks and warehouses with each other and with possible factory sites in the rear, planned for industrial development. It is intended that each of these terminals, including as long a stretch of waterfront as is locally available, shall be made accessible for car floats and steamers, with the ultimate expectation of connecting them together by freight tunnels under the harbor waters.

The physical features of New York Harbor are so large as to make the present problem of connecting the four great divisions—New Jersey, Manhattan, Long Island, and Staten Island—peculiarly difficult. But these present difficulties afford the future opportunities for the progressive commercial and industrial organization of the port, permitting apparently unlimited scope for expansion. Properly organized, New York will continue, not only the greatest port, but also the greatest manufacturing city. Proper planning and utilization of natural opportunities will enhance realty values and tax income and by so doing will make possible the rapid development of public and private improvements of all descriptions.

It should be the policy of the City and State of New York to anticipate and provide favorable conditions for the employment of the industrial and commercial capital and population which is naturally attracted to the port. This can best be done by developing and progressively executing a comprehensive plan for waterfront terminals. Any such general plan can be carried out only by the City itself and involves a large degree of public control here as at other seaports. Private cooperation will necessarily follow about in proportion as the public spirit of the City shall provide the opportunities.

The State has, by recent legislation, granted the City the power to organize its waterfront. The Appellate Division of the Supreme Court will this year determine the precise amount of dock bonds which, in its judgment, are self-sustaining and so exempt from from the debt limit. This will constitute the ''Dock Fund,'' which should be used for waterfront improvements in conformity with the practice of other seaports. Finally, the Board of Estimate and Apportionment should promptly adopt a definite plan and policy for the development of the port.

Respectfully submitted,

CALVIN TOMKINS,
Commissioner of Docks.

September 26, 1910.

Hon. Calvin Tomkins,
Commissioner of Docks.

Sir: In obedience to your order, I have visited the principal harbors of Northen Europe, and beg to submit herewith a brief résumé of some of the impressions received.

The docking facilities in all of these seaports are arranged in almost the same way, by building permanent quay walls along the streams themselves, and by creating basins, with this same wall construction, in from the rivers.

On account of the extreme rise and fall of the tide, most of the harbors are locked off from the streams themselves, forming by the use of gates what are called wet docks. Inside the basins the necessary wharfage length for accommodating shipping is obtained by building projecting piers, such as surround the Island of Manhattan, only instead of piers on piles, the sides and ends of the piers are built of granite, brick, or concrete walls, between which is solid filled-in land upon which area are erected permanent brick or steel sheds.

The piers are usually made wide enough to accommodate a shed on each side, with sufficient room between them for at least two railroad tracks adjoining the side of each shed, and a wide trucking and storage place between.

There is thus a great similarity in all of this harbor construction, the point of variation being in the location and method of operation of these facilities and such appliances as hoisting cranes, in the handling and delivering of freight and cargo to and from railroad cars and trucks.

One striking feature to be borne in mind in any comparison between American and European methods is the size and shape of the freight cars both in England and on the Continent. They are all very small, with a capacity of from 8 to 15 tons, and the majority of them are open, or only platform cars. For this reason they are particularly adapted to the handling and loading at close door intervals from some form of crane. This fact has brought the hoisting crane into almost universal use, and this machine has been brought to a very high state of perfection.

There seems to have been no unanimity of opinion in the past as to the best relative arrangement between the location of the shed and the cranes, and even in the latest and most modern developments, extreme differences are noticed. Thus, in the magnificent shed, about being completed for the Holland-America Line at Rotterdam, the shed is placed directly on the face of the quay wall, leaving no room for railroad tracks, while the cranes run along the roofs of the sheds. This arrangement follows the older systems found in Liverpool and in many portions of the docks at London and Southampton.

On the other hand, the most modern sheds at Hamburg, Bremerhaven, and in those now being built on a very elaborate scale for

the French Line at Havre, the sheds are placed from 30 to 40 feet back from the face of the quay wall, leaving room along the front face for the wide tracks of the traveling cranes, and also room for one or two railroad tracks, the outside railroad track being often placed between the rails of and under the crane itself. This system has also been in vogue in all the older sheds at Antwerp. It is rather astonishing that some one satisfactory solution has not been reached, so that one standard could have been established, as the business handled and the end desired are identical.

With the shed close to the quay wall, and there are many such, both old and modern, the condition is about the same as obtains on the New York piers, except that the hoisting is usually done by traveling cranes, delivering cargo from the ship to the side of the shed instead of hoisting by ships' winches, or by dock winches, as we do. After being hoisted to the side of the pier, the methods are exactly the same as ours, as all cargo is then taken on hand trucks through the shed for temporary storage, or to the trucks or railroad cars in the rear.

Notwithstanding the extensive use of cranes in Europe, the same system as ours is pursued at many first-class docks by hoisting with the ship's booms only, not even having the additional help of the dock winches which are coming into demand and use in New York.

The other extreme of placing the sheds well back and leaving room for cranes and tracks in front is found at Bremerhaven, where the North German Lloyd have a magnificent shed system. Here the cranes run up and down the docks, built high, and with a long reach of boom, so that they command the full reach from the ship's hold, and by a circular movement load direct to two railroad tracks, or directly over to the doors of the shed, at will.

The operation of unloading a large steamer with these cranes impressed me as being the acme of the unloading art, six of them working in almost absolute silence, some loading directly into the open railroad cars, while others delivered to the shed doors, in an almost human manner.

All of these docks are built in a most substantial manner, because in most instances a firm foundation is found at a reasonable depth, so that no piling is necessary for their foundation work. The basins were created along the streams inside the deep water line where the rivers were developed longitudinally, and where it was cheap and easy to build these walls dry, below the depths of future dredging in front, or else they were built entirely dry inshore from the waterfront, afterwards letting the water in.

At Antwerp to-day gigantic extensions are in progress, built entirely inshore in a dry basin, by constructing masonry walls dry, digging out the basin, and afterwards letting the water in.

Directly opposite to this method, at Rotterdam an extensive

basin is being constructed by simply dredging out a basin in the river itself, with no locks, and building a wall around it. They have not had to encounter the difficulty in foundation work which exists in New York, where the only chance for development is in building out from a rigidly fixed line inside of which no basins or other encroachments can extend, out into the river where all kinds of bottom exist, and where piling in almost every instance has been an actual necessity.

A universal feature of the European quay wall operation is the fact that almost all steamers lie directly alongside these walls, whether of brick, concrete, or granite, without any fender system whatever. In some cases light fenders consisting of balls of rope are used, and in Antwerp was noticed a fender made up of small saplings cut into 10-foot lengths, intermixed with large sticks all bound together by wire rope into bundles about 6 feet in diameter. These are suspended over the walls from the mooring posts and afford effective but inexpensive buffers for the wearing of ships against the walls.

Another striking thing which they never do abroad, and which in New York we think is not only absolutely necessary, but mandatory on account of law, is the building and maintenance of all the quay walls with no backing log, or other barrier, between the street and water. All the walls are constructed entirely flush with the pavement. This condition exists over hundreds of miles of quay front which extends throughout the most populous cities, no thought having ever arisen that such a thing was at all necessary. The contrary condition exists in New York, where large sums are spent to maintain this backing log 1 foot high over every foot of open pier space and bulkhead wall front.

It is safe to say that New York is particularly blessed as against all the ports of northern Europe in the comparatively small amount of dredging necessary to build and maintain its wharves and docks.

Rotterdam is utilizing the excavated material, but in most cases it has to be taken on very long hauls to sea. Prodigious amounts of material have had to be dredged in order to obtain access to most of these cities, through cuts from 10 to 30 miles from the sea, besides excavating the basins which are the docks themselves, and in every case the shoaling formed by matter in suspension from these rivers and estuaries renders the task almost herculean and very costly.

The wonderful skill of the pilots, and the efficiency of the tugs themselves are noteworthy, especially in Rotterdam and Hamburg. Like the railroad cars, the barges and tows are small; hence the towing boats are small—in fact, big launches, and the way they are handled in steering tows in and between this congestion of river traffic is simply marvelous. So much depends upon the quick delivery of these small barges to the steamers anchored to dolphins in the streams or basins, and upon the landing made by passenger

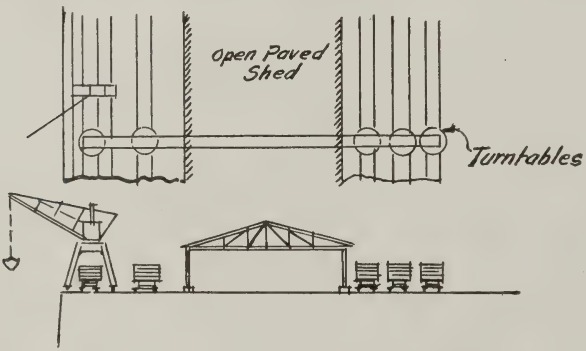
steamers to the various landing stages, that the excuse for their methods lies in this necessity; otherwise, it all appears to be foolhardy, and continued river accidents are only avoided by a combination of extreme skill, combined with good luck.

This whole trip of inspection has been one of extreme interest to me, affording an unusual opportunity for receiving much instruction in a personal way.

The conditions existing on our water front, and the opportunity for weighing the difference between domestic and foreign methods looking to opportunities for betterment will no doubt be of benefit to the city in the future.

The small sketches of the various ports are placed in this report to show in a simple manner and at a glance the relation between the rivers or open harbors themselves, and the constructive basins, together with the projections for additions where they are being built or where they are merely suggested.

The sketches of sheds are merely submitted as taken on the



ground to show the different types which represent as many differences of opinion as to the proper relation between the dock shed, railroad tracks and the loading cranes.

So much has been written in description and in statistics of these ports that I will only submit some notes of conditions taken at the time and in the order of visit.

ANTWERP.

The River Shelde is developed to its fullest extent by the construction of a quay wall throughout the entire length of the city, the bulk of the wharfage capacity being derived, however, from the construction of enormous locked-in wet basins, the tidal range being 16 feet neap, and 20 feet spring tides, two tides occurring in twenty-four hours.

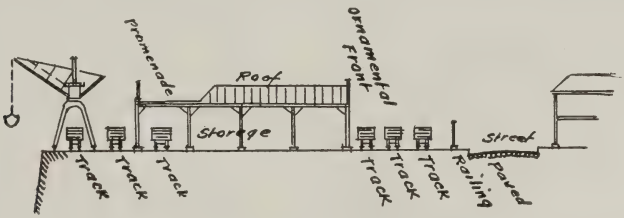
The feature of the port is the open sheds built somewhat away from the quay wall, leaving room in front for the wide traveling crane track and two or three railroad tracks. The crane usually

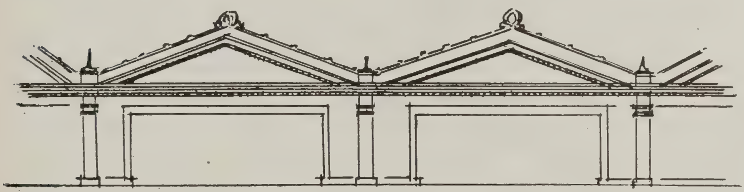
spans the outside track, and the outside railroad tracks are connected with the tracks in the rear of the sheds by turntables at intervals, which carry the loaded cars from the front tracks to the rear of the shed and then to the track system.

The sheds are usually of corrugated iron, with corrugated roofs, some of the newer ones being built of brick with steel rolling doors.

The quay walls have been built in concrete faced with brick and granite coping, the newer ones of concrete, all taken to firm foundation without the use of piles.

Almost all the sheds are far enough from the front face of the quay wall to permit from one to three tracks, and the crane on tracks, which transports the cargo from ship to cars on the front tracks direct, or else to the doors along the face of the shed. The cargo is then placed on hand trucks and wheeled immediately to





ORNAMENTAL·FRONTS·

the cars which stand on the tracks in the rear, or is left in the open sheds for short storage, or until such time as it is taken away by trucks to the warehouses.

Some of these open sheds are made safe for cargo in bond, by placing an iron cage around the sides.

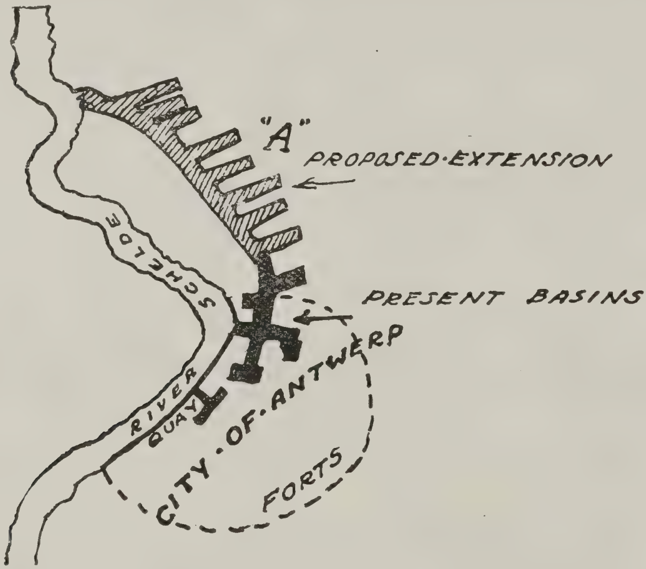
The distances between quay wall, crane and shed exit vary, but always on this same principle, the sheds being usually open on the sides and of simple construction.

Many large warehouses are found around the quays, all with ample railroad connection, or short-cut connection with the quays, and often extend five to six hundred feet through to the nearest street. Some of the latest and most modern warehouses have been built of reinforced concrete and in an elaborate and costly manner.

Along the river front proper the same system is followed, except that inasmuch as a marginal street exists in much the same relation to the river as West Street in New York, an attempt has

been made and quite a satisfactory solution found, by which the river front traffic is isolated from the business of the street.

As shown on preceding page, the shed is placed in from the quay face far enough for the traveller tracks and two railroad tracks, the outer 20 to 30 feet of the shed itself being transformed into a promenade for people by using the roof. It is in the rear and out of the way of the cranes and railroad tracks, making a fine promenade for recreation, and does not in any way interfere with the business of the waterfront. The rear face of these sheds along the street fronts is treated in an architectural manner, the facade created being pleasing to the view and at the same time forms a barrier which conceals the objectionable features of the waterfront opera-



tions. Finally, a fence entirely separates access from the marginal street to the railroad tracks, connections from the streets to sheds and railroads being provided at intervals.

Should the future growth of New York demand that commerce invade precincts which are now reserved for other purposes, some such treatment may well be followed.

It is easy to imagine how some sections of New York, where any attempt of this kind would be tabooed and considered a desecration, could be treated in some such manner, which would not only redound to the infinite benefit of the city from a commercial point of view, but which would perhaps bring the real waterfront into a closer and better relation with the poor people who really need it, from the standpoint of health.

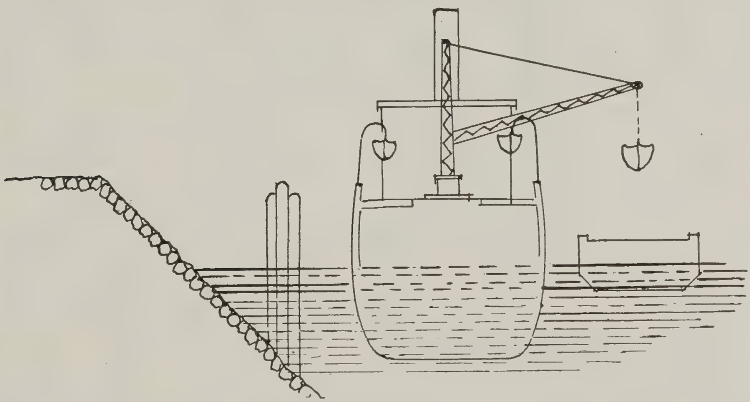
Locked-in dry or graving docks abound here, which accommodate

the largest steamers coming to the port; and a new and enormous dry dock is being built.

Two enormous new basins, each much larger than any of the old ones, have been recently built, and are now crowded to their full capacity. It was necessary in building these new basins or docks to cut through the fortifications which surround the city, and although obsolete, new breastworks and ditches have been extended at great expense around the newly created basins.

Realizing the inadequacy of these new and large additions, the city commenced about one year ago to build a new and really tremendous addition to its dock system in the form of an addition and continuation of these new basins, shown at "A" in sketch on opposite page.

It reminds one in its present condition of an enormous reservoir in construction, showing an enormous excavation for the surround-



ing quay walls, which are being built dry. After the walls are finished, locks will be built to connect the new area with the nearest basins already built, after which the new basins will be flooded and the whole excavation for the new basin be completed by ordinary dredging with floating dredges.

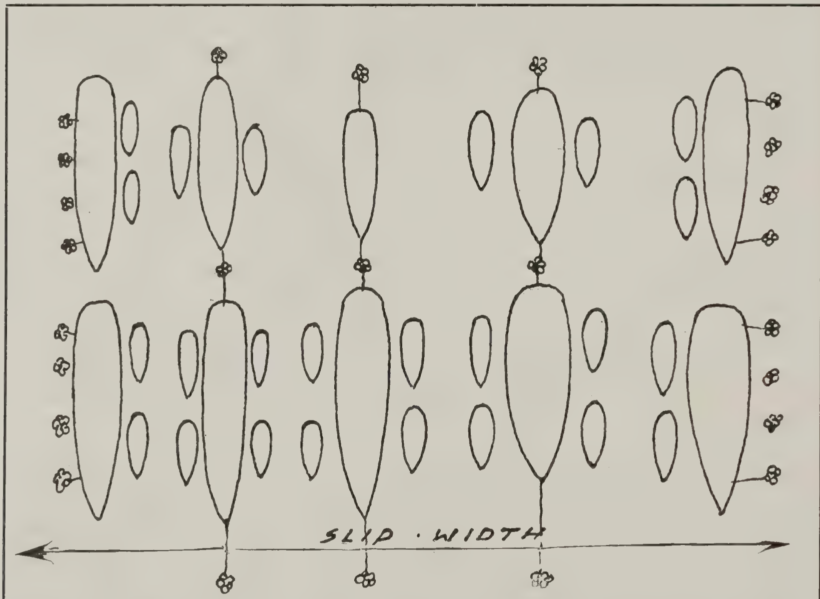
This addition to the harbor is being carried on in the most extensive manner, and is intended to supply the demands for many years to come.

ROTTERDAM

The River Maas is utilized on both sides throughout the City of Rotterdam for wharfage on the quay walls which have been constructed in a permanent manner of brick, rough rubble in cement, or of concrete with granite coping. The greater part of the wharfage, however, is carried on in basins where sheds have been built around them. The range of tide being insignificant, no locks are necessary, all wharfage and dock development being carried on from the open river.

Besides the permanent quay walls where the larger steamers berth for loading and unloading, there are many miles of sloped walls, cheaply built by paving the slopes of the excavated basin, after dredging, then driving clusters of piles at the foot of the slope where water is of sufficient depth to berth vessels of ordinary size. At such berths as these much business is done, as the vessels are kept here for storage and lay-up, as an awaiting place for dock space, or are often loaded and unloaded to and from barges which lie outshore, or on the basin side.

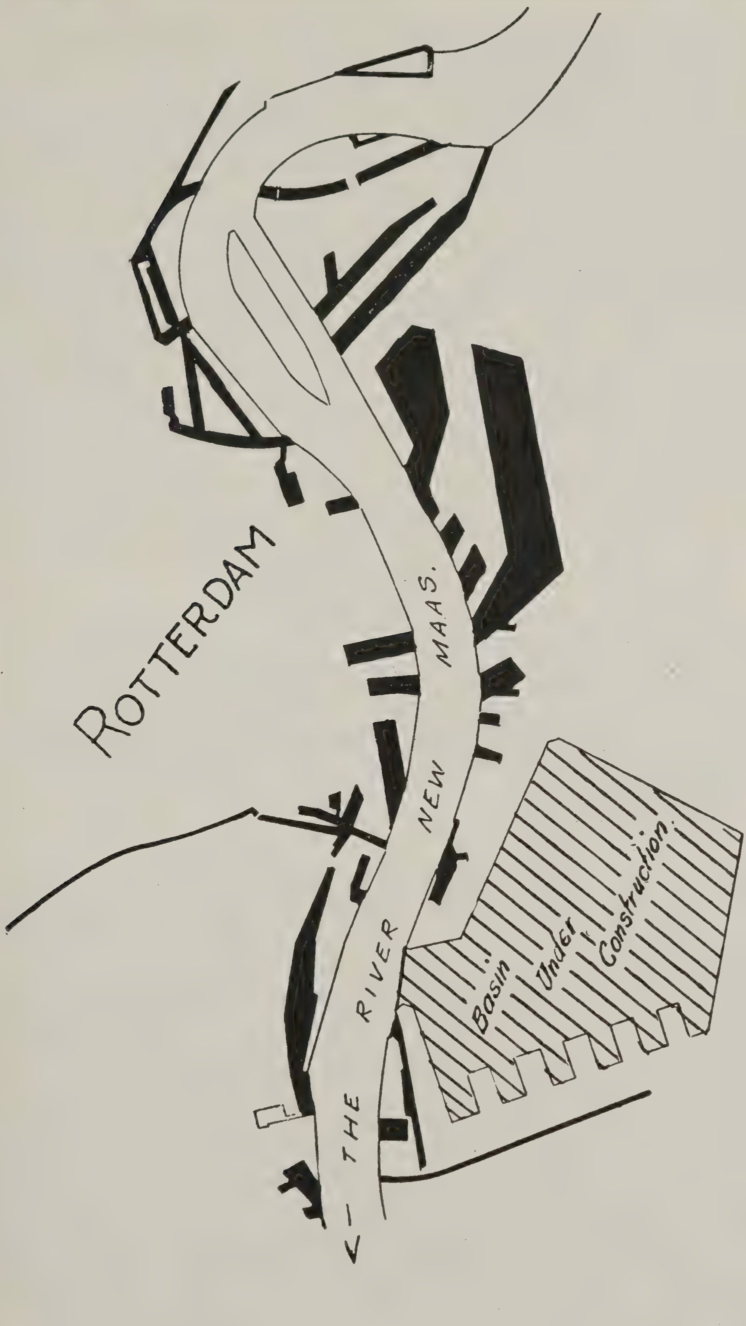
The large and wide basins are utilized to their fullest extent by driving these pile clusters at intervals through the basins longitudinally, where they act as mooring places for the transfer of cargo



from barges or canal boats. In some of the basins as many as five lines of these mooring dolphins exist, where large steamers are loaded from floating coal conveyors in the middle of the slip and loaded with grain from floating conveyors or elevators, and at the same time take cargo from barges, as above shown.

The city is well supplied with these floating conveyor machines, and with floating derricks of great capacity, and while there are no graving or dry docks, there is a number of large floating dry docks which can accommodate the largest steamers using the port, except the largest of the Holland-America Line which is obliged to go to Southampton for docking.

The sheds at Rotterdam are built at various distances from the quay face, leaving room usually for the operation of movable cranes which reach to one, two, or sometimes three tracks in front of the

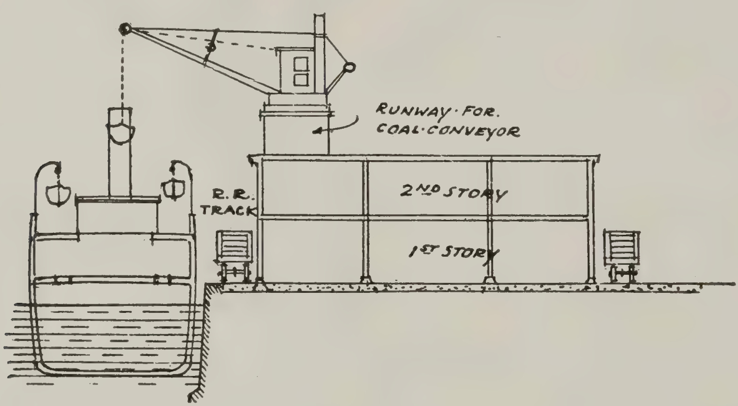

ROTTERDAM
THE RIVER NEW MAAS.
Basin Under Construction

shed. The sheds which adjoin the last or inshore track are built usually of corrugated iron, are enclosed and extend to the car tracks or unloading platform for trucks in the rear.

The cargo is handled by cranes from the hold of steamers to cars on front tracks, or to the door of the shed, thence by hand trucks for storage or to the railroad cars on the rear tracks, or to the tracks for distribution.

While this system has been the universal custom in the past, the newest and what is considered to be the most modern system of sheds is now being constructed for the Holland-America Line. The city builds the basins and docks, and the company is erecting its own shed under a twenty-five year lease, with certain terms for renewal.

The arrangement and location of these sheds is being changed from the universal past policy of the port, by placing the face of the shed close to the quay wall. This arrangement leaves only



room for one track on the outside of the shed; the cranes themselves running on the top of the shed at a high elevation, so that the booms will reach over into the holds of the largest steamers, and swing the cargo direct to the doors at the face of the shed. Here the cargo is taken on hand trucks for storage on lower story, to conveyors for storage in second story, or to the rear of sheds to truck delivery or to railroad cars.

These sheds are being constructed of reinforced concrete with steel rolling doors in the most substantial manner, and separated about every 250 feet by concrete walls into fireproof compartments with only small doors for access from one to another. At each partition the building is entirely separated from the part adjoining it by a 2-inch wide joint, forming entirely isolated structures at these intervals for expansion, settlement, etc. These sheds, when completed, will probably be the most perfect, elaborate, and costly so far undertaken in Europe.

The type in cross section is as shown in the sketch above.

The city of Rotterdam is now constructing a most extensive addition to its harbor facilities, by dredging an enormous basin which will cover several hundred acres. At present a large area has been dredged, and while only in its first stages, as soon as a sufficient area has been excavated to accommodate a vessel, the berth is engaged and occupied. In its first or present stage the sides of the basin or dock are simply sloped off by the dredge, while piles, clusters or dolphins are driven where the water is deep enough to accommodate a vessel, and to which they tie up. The intention is to later gradually build the permanent quay walls, and then finally deepen the whole basin by ordinary dredging so as to accommodate the largest steamers anywhere within it.

The excavated material, which is transferred to scows by the bucket conveyor dredges, is immediately taken across the river with a very short haul and deposited, on the low marsh lands owned by the city, by pumping after the material has been raised by bucket conveyors from the scows. These marsh lands are now valueless, but when the basin has been excavated many hundreds of acres of fine land will be created upon which the city is now expanding and in future will extend.

HAMBURG

This city, now of about 800,000 inhabitants, has constructed a wonderfully efficient system of docks or basins, after contending against the fearful handicap of having to dredge the River Elbe for 64 miles at its entrance on the North Sea. This disadvantage is somewhat mitigated when compared with other ports of Northern Europe, in that the range of tide is slight, hence no locks have been necessary; all commerce being conducted on and from the open river.

Both sides of the river have been completely built upon throughout the city, but the greater amount of the wharfage room is acquired by the construction of enormous basins; these basins running everywhere, together with the great number of canals which interline the whole city, bring the wharfage of the port into the heart of the city itself at every part of its area. The wharfage is obtained by the steamers berthing longitudinally alongside of the long quays, which have been built in a permanent manner of stone, brick, or concrete on piles.

In many cases in the outlying districts the sides of the quays are not so finished, but the sides of the banks have been sloped off and paved, the vessels being tied to dolphins driven where the water is deep. This system, as in Rotterdam, is also used to an enormous extent in Hamburg, of utilizing the basins and streams by lines of dolphins, where vessels can be loaded and unloaded from barges. Many of the smaller basins remind one of Newtown Creek, with its factories and storehouses, except that here the railroad tracks and connections are always in evidence.

Hamburg has no graving or dry dock, but is well equipped with many enormous floating docks which accommodate all the largest

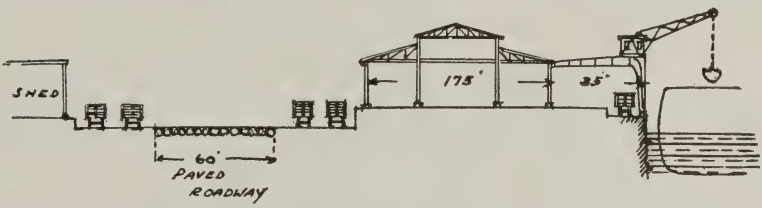

CITY · OF · HAMBURG
ELBE
RIVER
DOCKS
HAMBURG·AMERICAN LINE
PASSENGER·STATION
LANDING·PLATFORM
HAMBURG AMERICAN LINE

steamers using the port. One is being built for a capacity of 40,000 tons. The hoisting of very heavy articles is facilitated by taking the steamers to stationary cranes of huge lifting capacity, while in other cases the hoisting of great weights is performed by enormous floating cranes.

The Hamburg-America Line has by far the most complete and elaborate system of docks, all built by the State, including quay walls, sheds, and cranes, of which there are many paying in rent about $325,000 per annum.

The docks where their steamers are berthed lie on the opposite side of the river from the city proper, the passengers being received at a station in the city on the river bank, which is connected with the landing stages by enormous steel gangway bridges. Small steamers transport the passenger from the station to the landing stages across the river about 1 mile away, which are near the docks where steamers are berthed.

This Hamburg-American passenger station in Hamburg is a most elaborate and expensive structure, extending some 700 feet along the river side. It is of the finest cut granite throughout, and is relatively much more expensive than the shed structures



built by the city of New York at the Chelsea Piers. The steamers dock at three enormous quays, with well constructed sheds about 175 feet wide and 2,500 feet long. The sheds are built back from 30 to 35 feet from the face of the quay walls, leaving room for the tracks of large electric cranes which then span an outside railroad track, over a handling and storage space to its inside leg which is a part of the column of the shed structure. The crane then lifts from steamer, and by being run along the cross-ways inshore has a range from steamer to outside railroad tracks, over the outside span, of 30 to 35 feet, and up to the doors of the sheds. The cargo is then taken on hand trucks through the shed to the railroad tracks in the rear, or to wagons for other deliveries to warehouses; other parts of the cargo are left in the sheds for temporary storage until taken away in a few days by wagons or cars.

The cranes unload a ship quickly, thus relieving the berth, while the sheds are capacious enough to accommodate a steamer's cargo, until it is hauled within some reasonable time.

These steamship basins or docks, which represent the best, have been built about seven years, and are adequate for the needs of this great company.

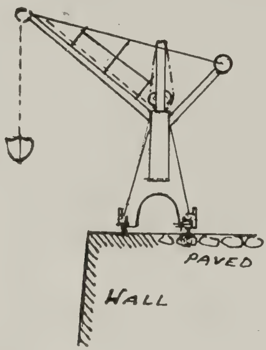
A large addition to the basin or dock system is constantly being considered by the city, but nothing is being done in construction.

BREMEN

The harbor for big steamers at Bremen is at Bremerhaven, at the mouth of the River Weser, on the North Sea.

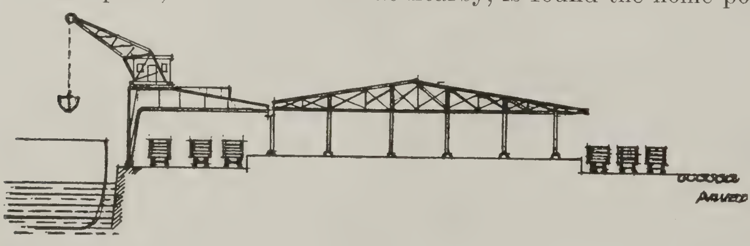
At both places locks are necessary on account of the lack of depth in the outside streams, although the tide has a range of only about 8 feet.

Bremen has an extensive dock system and it is being extended, but on account of the lack of depth in the river it is only for comparatively small steamers; while at Bremerhaven, the big



steamer port, an extensive harbor has been artificially created, and is now being extended, so that if the very ambitious plans for the future are carried out, Bremerhaven will become one of the big ports of the world.

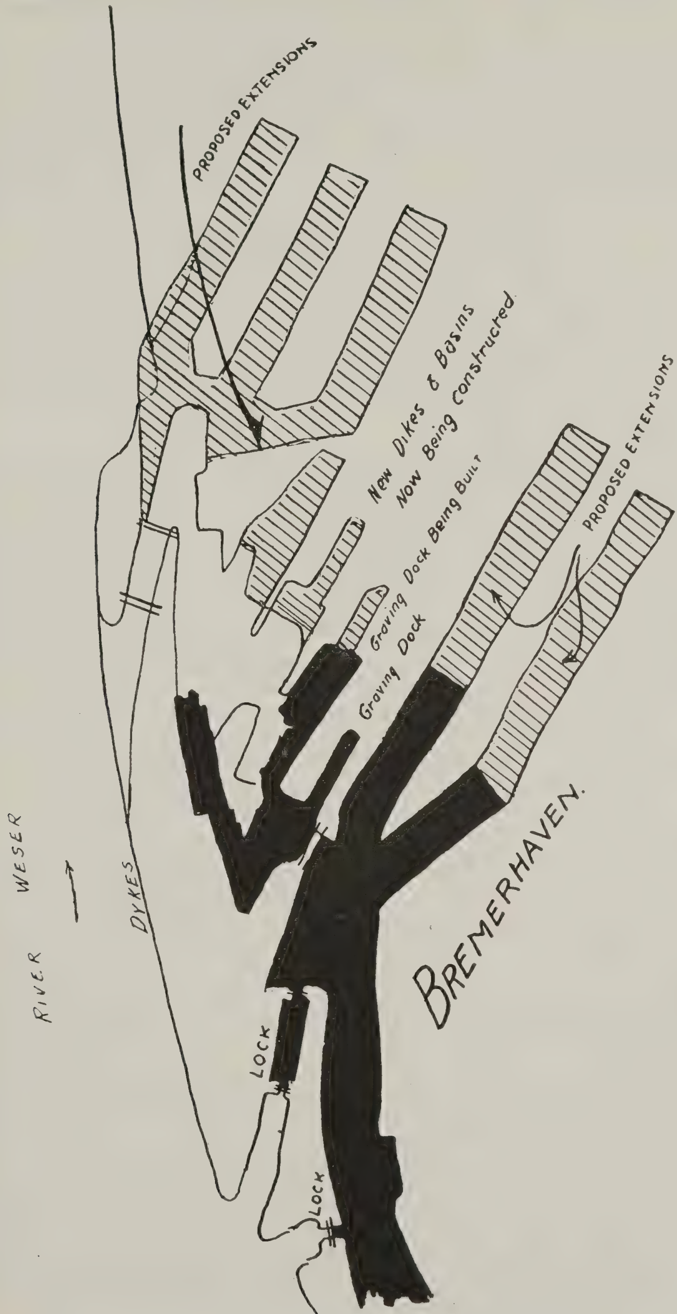
At this place, and at Gestemunde nearby, is found the home port



and headquarters of the North German Lloyd Steamship Company, including the landing places for passengers, and all cargoes, together with their machine and repair shops, which are extensive.

The State has constructed an enormous artificial harbor by a dike system from the river, which is here almost the North Sea itself; the entrances to the excavated basins or docks being effected by regular granite or brick locks.

Many of the quay walls are open, with no sheds, but simply an elevated outside platform upon which run traveling cranes, transporting the cargo from the ship to the wagons which back up to the platform.


PROPOSED EXTENSIONS
New Dikes & Basins
Now Being Constructed
Graving Dock Being Built
Graving Dock
PROPOSED EXTENSIONS
BREMERHAVEN.
RIVER WESER
DYKES
LOCK
LOCK

For the steamship company the State has erected several enormous long sheds about 200 feet wide, all constructed in a very substantial, but not elaborate manner, with corrugated iron sides, wooden roof trusses, and tar and gravel roofs.

The electric cranes are built so that the outside leg rests on a truck running on a rail along the quay wall, while the inside leg rests on a truck rolling on a rail secured to the shed columns. This enables the crane, which is built very high and of an enormous reach, to pick up cargo from the holds of big steamers and by a simple circular movement drop it into the cars on the three lines of railroad tracks over which it runs, or to the door of the sheds, all from its original pivotal position on the outside of the quay.

A careful inspection of the work of unloading a big steamer by these silent hands, working in an apparently almost human manner, would indicate that this method exemplifies the perfection in crane work as applied to handling vast numbers of trucks and at the same time to the shed itself.

This company uses a system of loading a steamer at certain berths, simultaneously, by hand delivering into wagons on the quay side, and on the basin side from barges; the hoisting being done by the steamer's winches and booms. For unloading, the steamers are taken to the long shed where the cranes are used as shown in sketch on page 70.

The port is well supplied with several first-class dry or graving docks, the largest, or Kaiser Dock, being capable of docking their largest steamer, the *George Washington.*

Another enormous dry dock or graving dock, 260 meters (845 feet) in length, is now being built for the largest steamers. It is being constructed by dredging over the flat lands, and will, together with a huge extension to the basin system which is also being dredged out, be completed in 1912. Land has been acquired, and the project is now receiving active and real consideration for a further gigantic extension to the quay or basin front, which will add about 10 miles to the already constructed 5 miles of wharfage length, and which is expected to accommodate the commerce for the next fifty years.

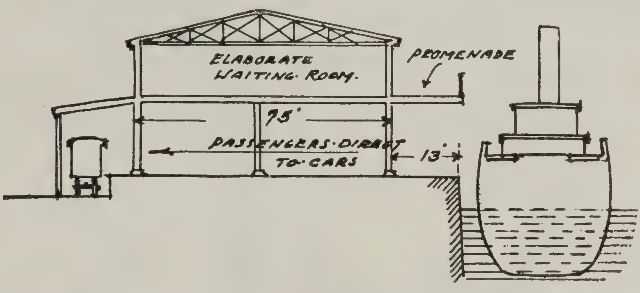
HAVRE

The harbor at Havre is formed by building a breakwater out into the sea, and up the River Seine at its mouth, locked-in basins being necessary on account of an extreme rise and fall of the tide of 25 feet. Notwithstanding this, some steamers and small craft use the quay walls which exist outside the locked-in basins, lying aground at low water.

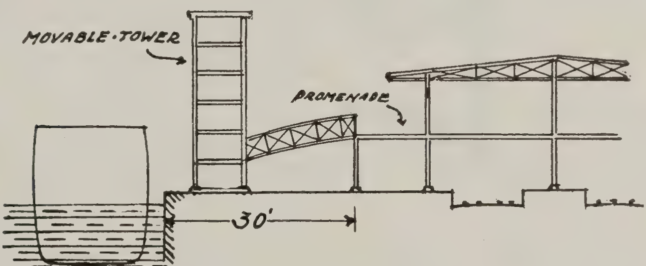
All the quay walls are old, well built in some kind of masonry, and in many cases are not shedded, but paved up to the face of the wall for a general unloading business. In general, the sheds have not been extended to such an extent as in other ports, and are of an inferior construction. Cranes also are not so much used as in other European ports.

The port authorities build all quay walls, and all sheds, except that in one instance the French Line has built its own shed.

The above statement that the sheds are of an ordinary quality does not apply to passenger sheds now in use by the Compagnié Géníral Transatlantique, and to the fine new shed now under construction for that company. The present shed for passenger traffic is very complete, and is shown in the following cross section:



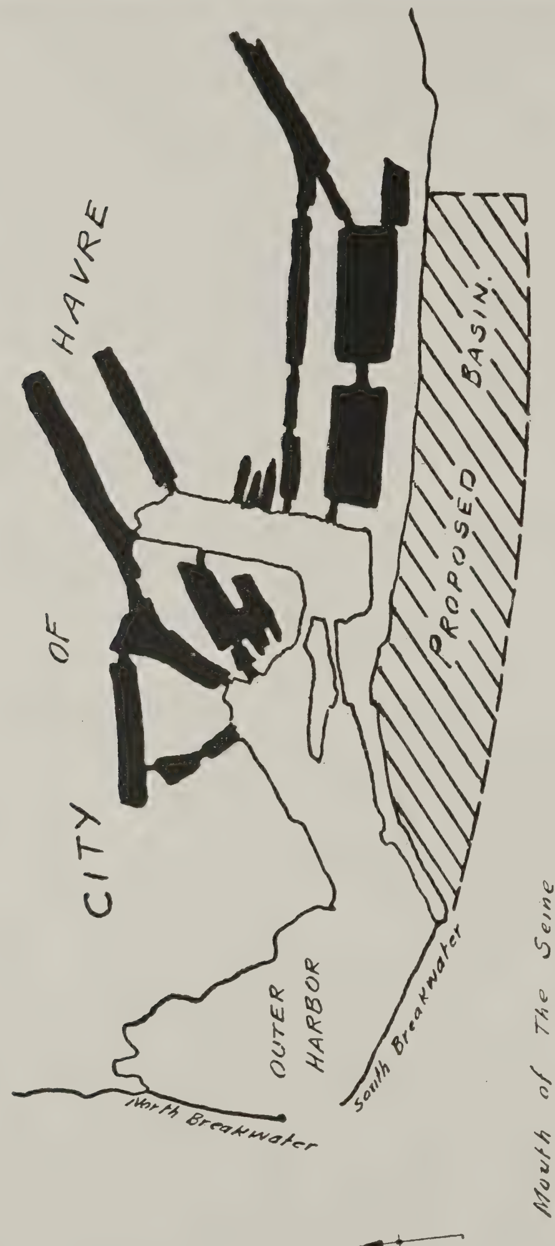
The new sheds for this company are being built outside the locked-in basin system, so that passengers may be landed at any stage of the tide, the steamer afterwards being taken inside the basins to other landings and sheds for unloading and loading the cargo. This structure is shown in the cross section below.



The whole structure is of concrete placed about 30 feet back from the face of the wall, on the outside of which run two extensive and high travelling towers, which are moved to the necessary gangways, and passengers are landed in the second story by gangway bridges.

On the outside of the shed on the second story is a wide and grand promenade for passengers, while inside from the promenade is an elaborate and very tastefully decorated waiting room. The lower story of the shed is divided into two separate concrete platforms, and accommodates four lines of railroad tracks, with every appearance of a regular passenger railroad station. The shed will soon be completed, and the French Line will probably have the most complete and elaborate passenger landing in Europe.

All the sheds in the port have direct connection with the rail-


CITY OF HAVRE
PROPOSED BASIN.
OUTER HARBOR
North Breakwater
South Breakwater
Mouth of The Seine

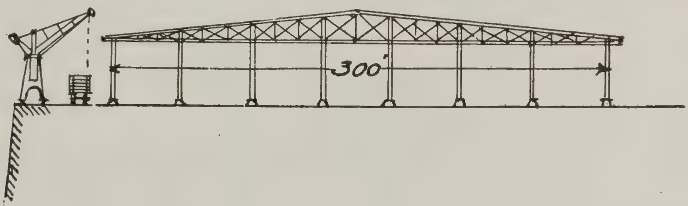
roads, but most of the cargo handling is done with ship winches and booms, although the French Line has many floating and travelling cranes for heavy cargo and for coaling.

The coffee warehouses are marvels in extent and capacity, and the new sheds for the cotton trade are of immense size, 300 feet wide and 2,600 feet long, as shown in cross section below.

These sheds are built so that all the sides are rolling doors. The roofs are tar and gravel, with flat glass skylight scheme, every 70 feet, running entirely across the full width of the shed.

The port is well supplied with fine graving or masonry built dry-docks, which accommodate the largest steamers using the port. For the new steamers now being built for the French Line, dry-dock accommodations will have to be used, however, at Southampton.

Although possessing an ideal location for a harbor, not only on account of being practically located on the ocean itself, but from its command of the Seine Valley, Paris, and the whole Continent direct, Havre seems to be neglected in trade. Notwithstanding this apparent lethargy, extensive and very bold schemes are pro-



jected for increasing the capacity of this harbor to three times its present size, to accomplish which the building of new breakwaters will be necessary upon the extended lines of those already built, out into the open seaway, forming a new and gigantic basin for more wharfage.

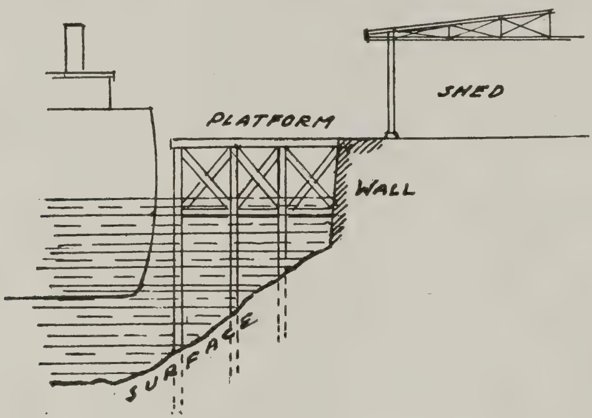
No construction has yet been commenced, although the project seems to have at last assumed definite shape.

SOUTHAMPTON

Admirably situated in one of the finest natural harbors, Southampton no doubt has a future through its dock system, on account of its easy approach from the open sea, its nearness to London, and on account of the fact that it possesses a double tide, that its open quay berths are accessible for the largest steamers at any stage of the tide, not being at all dependent upon locks.

Another attractive feature is that at this port the well-equipped masonry, graving, or dry docks are subject to use by direct transfer of ship from the open sea, without first having to go through other locked-in wet docks to enter them. The saving in time and bother is thus apparent. These graving docks accommodate the largest steamers of the International Mercantile Marine Company.

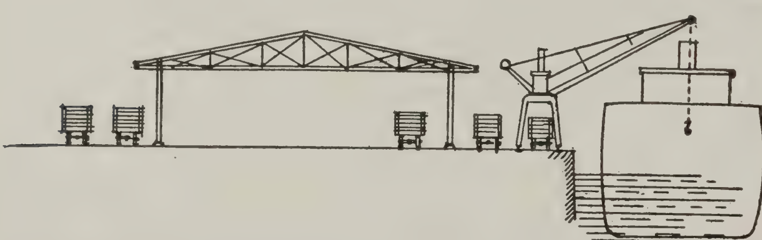
Part of the business of the port is conducted along the outside quay walls, which parallel what is practically the open harbor, and part of it along the sides of quay walls in the excavated basins, of which there are four large ones. All these walls and basins are old, having been commenced in 1838, only one new one having been built, now nearing completion. This basin is being built of



concrete in place, dry, in open cofferdams, carried down to a firm foundation in clay, without any piling.

One of the quay walls in the open harbor has been built inshore from its regular face line where the water is shoal, to avoid expense.

The necessary depth for steamers is obtained by berthing them

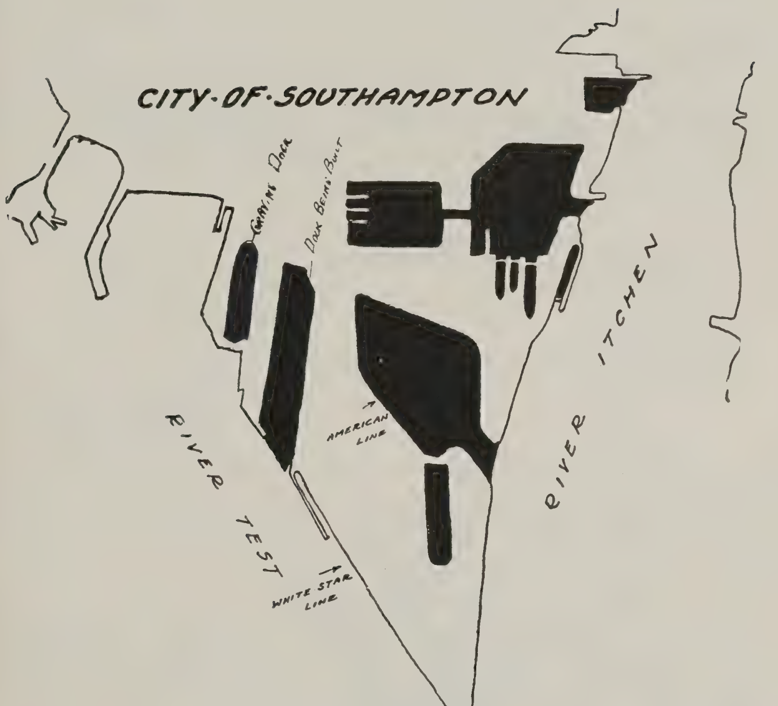


outshore from the wall, alongside a heavily built platform, as shown in cross section in the upper illustration.

The White Star steamers dock on a quay in the open harbor, using a few large travelling cranes for heavy cargo placed on tracks at the edge of the quay wall, commanding one railroad track under the crane itself, and another between it and the shed, as well as the doors of the shed. The sheds are of ordinary construction, as shown in the lower illustration.

This company has 32 feet at M. L. W. spring tide.

The American Line sheds for passengers and freight are of inferior wooden construction. This type of grading up the floor


CITY·OF·SOUTHAMPTON
GRAVING DOCK
DOCK BEING BUILT
AMERICAN LINE
RIVER TEST
WHITE STAR LINE
RIVER ITCHEN

to the rear for goods from far railroad tracks is used in many of the other sheds, and is shown in cross section below.

LONDON

The harbor of London commences in the city proper, near the Tower of London, and continues practically down the river for about 22 miles to Tilbury. The principal docks, however, are the Victoria and Albert. The Tilbury docks are about 22 miles down the Thames.

Until recently the docks were owned by a number of joint stock companies, but since 1908 a new port authority has been instituted for a public and city control of the whole dock system. In anticipation of this change to city ownership, the private companies had for a long time expended but little money, and had allowed the docks to deteriorate, so that now the city has much to do in order to bring them up to a condition of efficiency.

Because of the 17-foot range of tide, all of the docks in the

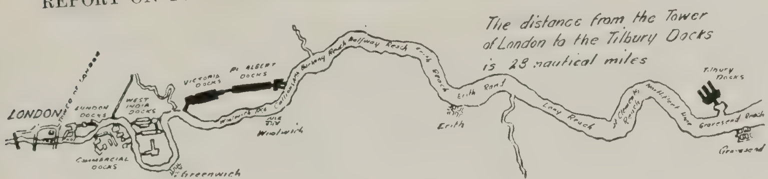
Thames are locked-in from the river by artificially constructed basins, all built at great expense, all old and some of them very old.

The Victoria Dock

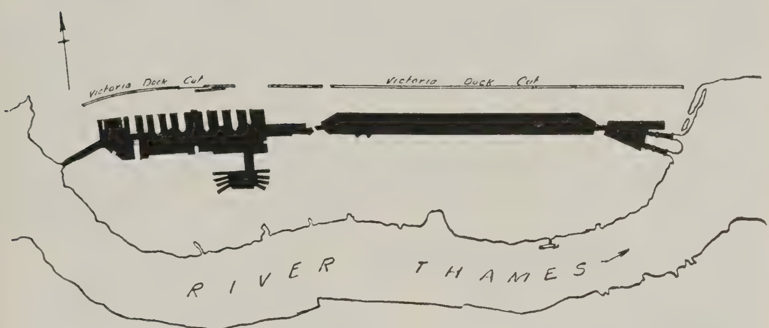
This is the nearest big basin to the city. It is old, and built of wood. From one side of the basin ten solidly filled-in piers project, along which some cranes have been placed, the hoisting being done mostly by ships' winches, however.

The Albert Dock

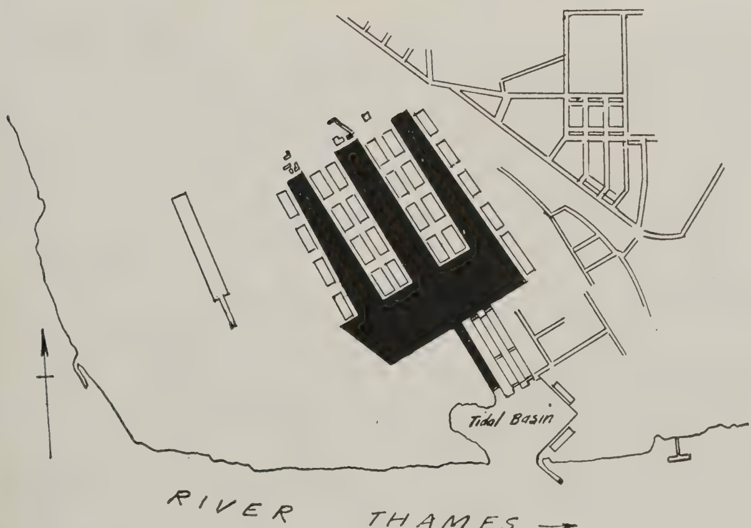
This basin adjoins the Victoria farther down the river and is connected with it by a lock. It is about 1½ miles long, built up on each side for its full length with sheds of light, cheap construction, with corrugated iron sides and roofs. These sheds are built about 35 feet back from the face of the quay wall, and leave room in front for two railroad tracks, besides the tracks for the travelling cranes. The goods are either delivered to the cars on the tracks by cranes or into the sheds at the door openings, for temporary storage, or else are taken on hand trucks through the sheds to the



LONDON DOCKS.



VICTORIA & ALBERT DOCKS.



TILBURY DOCKS.

railroad cars or wagons in the rear. This arrangement is shown in cross section below.

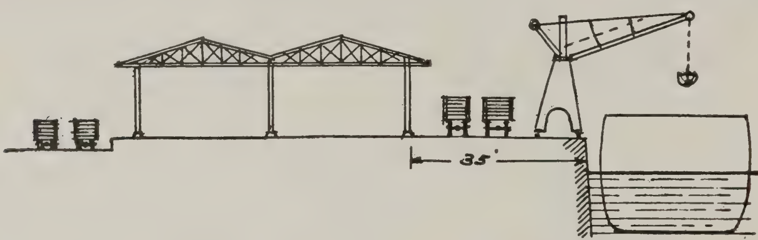
The arrangements for unloading and the relation between sheds, cranes, and tracks seem to be good at this basin.

The quay wall, over fifty years old, is built of concrete, carried down through a marsh which existed over this whole section, for a depth of 28 to 30 feet into a gravel, which affords a foundation for the walls without any piling.

The concrete face is in fine condition, the sides showing the extreme effects of wear and abrasion from vessels after years of direct contact and exposure, owing to the absence of any fender system. These docks are well supplied with floating, as well as travelling cranes, some of them of 50 tons' capacity, and with floating coal conveyors.

Two concrete graving or dry docks in concrete are in a fine state of preservation, although built twenty-six years ago.

In some of the older basins in London, many of the piers which had formerly been constructed have been removed, while the use



of those remaining is not popular with the port officials. After having many times expressed surprise at this condition, I learned that in many places, especially in London, 80 per cent of an ordinary cargo is deposited into barges on the basin side of the steamer, while only 20 per cent goes into the shed or railroad cars. For this reason, the wide span is needed in the basin for the operation of these barges, which is not permissible in the width of ordinary slips.

The port of London authority is confronted with a serious problem in the matter of proper repairs to the walls and lock gates at the Victoria and Albert docks, and at the same time keep them in condition, for these docks are the main big ship delivery points for London. At present they show the effect of long use without proper care in maintenance.

An extension to the wharfage capacity of London is now being actively considered and will be a duplication of the Albert Dock, as shown on sketched map.

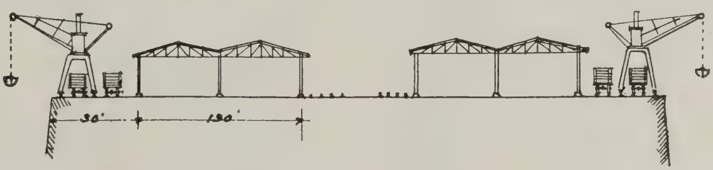
Tilbury Docks

The big steamers using the port of London dock at the Tilbury Docks, which lie about 40 miles from the mouth of the river proper,

and about 22 miles from the center of the London district. On account of the 18-foot tide, and the exposed condition in the Gravesend Reach at Tilbury, an outside tidal basin is maintained in order that steamers may await the tide to enter the regular locked-in basins, where almost all the business is done.

The wharfage is obtained in the main basin from three main branch or basin slips 1,800 feet long and 300 feet wide, with piers built in between them, similar to the pier system in New York. A cross section of one of these piers is shown.

The piers are built very wide, as shown, with sheds 30 feet inside the face of the quay wall, with room for two railroad tracks, and tracks for travelling cranes. The track system reaches the rear of the sheds, then, after an open space, the whole arrangement is duplicated for the opposite side of the pier. This takes up a very wide space filled in behind permanent quay walls, and is more like a regular piece of upland than a pier. The quay walls are of concrete, sometimes with brick facing and granite coping, about twenty-five years old, extending down through the mud to a gravel foundation without any piling. No fender system is placed on



these walls, the steamers lying directly alongside the brickwork. At the time of my visit the steel hull of the Atlantic Transport Liner's steamer *Mesaba,* lying alongside of one of these quay walls, was rubbing against the brickwork without any kind of intervening fender or buffer.

The sheds are cheaply constructed of light steel trusses, with corrugated iron sides and roofs. They are used simply for sorting and short storage, as most of the cargo here, as well as at the Victoria and Albert docks, is taken on the slip sides in small barges and canal boats. The sheds and railroad tracks are, therefore, not used to anything like their capacity, and are used less each year because of the preponderance of the barge method of transportation.

The Atlantic Transport Line's passenger landing stage is simply a covered shed, through which passengers walk to take the train for London, with no waiting room accommodation whatever. To illustrate the use of these sheds, one hour before the sailing of the *Mesaba* not a box or parcel was to be seen in any portion of the shedded area, everything being absolutely quiet awaiting the arrival of the London train.

These docks will require considerable attention in the matter of repairs by the new régime of the Port of London authority. At the present time, they appear to be quite overburdened with busi-

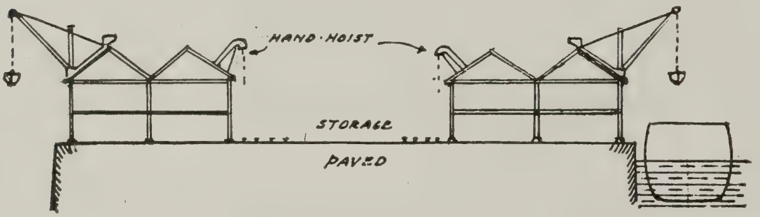
ness, but it seems that there is ample room for extension of the system, as the city owns the necessary land adjoining the present docks.

Two large concrete and brick graving docks accommodate the largest steamers entering the port. These dry docks are so built that they may be separated into two compartments for smaller vessels.

LIVERPOOL.

This great dock system has been created by building enormous walls along the river in order to create basins for the reception of vessels through the tide gates, against a tide range of about 26 feet.

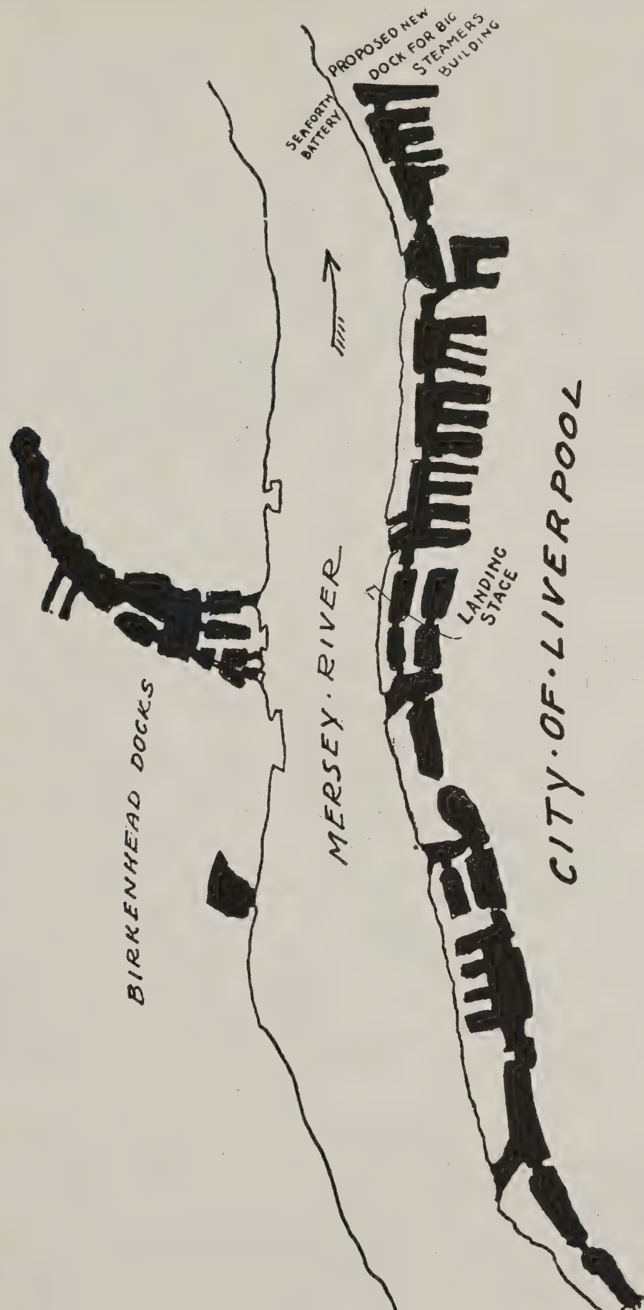
The whole dock system of the port is now under the Mersey Docks and Harbor Board, consisting of twenty-eight members, under whom are the various departments and offices for the control and maintenance of the entire system. The Liverpool and Birkenhead docks now have a total area of 582 acres, and a length of quay of 36 miles. To obtain this length the river had to be



fully developed for 7¼ miles. Sixty-three wet docks and nineteen graving docks make up this development, with 370 ordinary cranes, and many large high-power stationary cranes. Floating coal conveyors facilitate the coaling of vessels direct from railroad cars to ships.

Of late years the quay walls have all been thoroughly built of concrete with granite face, the walls being carried through the mud to a solid foundation, with no piles. About half of the sheds have no hoisting devices, while fully one-half are constructed in a first-class manner of brick, with iron sliding doors. In many cases, as for instance the Cunard and White Star lines, these buildings are three stories high, all close to the quay wall, and are supplied with derricks which travel back and forth longitudinally on the roofs.

The berthing places in these basins are arranged much like the pier system in New York, except that the piers are built very wide, so that the sheds, built as they are with faces close to the face of the quay wall, receive cargo direct where it remains for short storage, or is conveyed by hand trucks through the shed transversely to trucks in rear, or shifted to a wide storage place in the building, which answers also for storage for the next building, which is a repetition of the foregoing, shown in sketch above.


PROPOSED NEW DOCK FOR BIG STEAMERS BUILDING
SEAFORTH BATTERY
MERSEY · RIVER
BIRKENHEAD DOCKS
LANDING STAGE
CITY · OF · LIVERPOOL

It is about three years since the last docks were constructed, and no new work is now being carried on. An extension basin 1,000 feet long is planned for the near future. This will be built as a continuation of the present docks, and will serve as a wet and as a graving dock, when occasion requires it, for the large White Star ships now building.

An elevated passenger railroad extends along the whole dock system, fenced off from the street proper, on what is really dock property, forming a marginal street. Underneath the elevated tracks are the surface tracks for freight which connect at intervals with the dock system. The whole dock system here seems to be admirably adapted to requirements of commerce. The large steamers load and unload passengers, during short periods only, at landing stages half a mile long. After unloading passengers the vessels are taken into their respective basins for unloading and loading. A peculiarity of arrangement here is that no tracks are on the quay proper, but are all in the rear of the sheds. There is a noticeable lack of railroad business, most of the cargo apparently being carried away to warehouses in trucks.

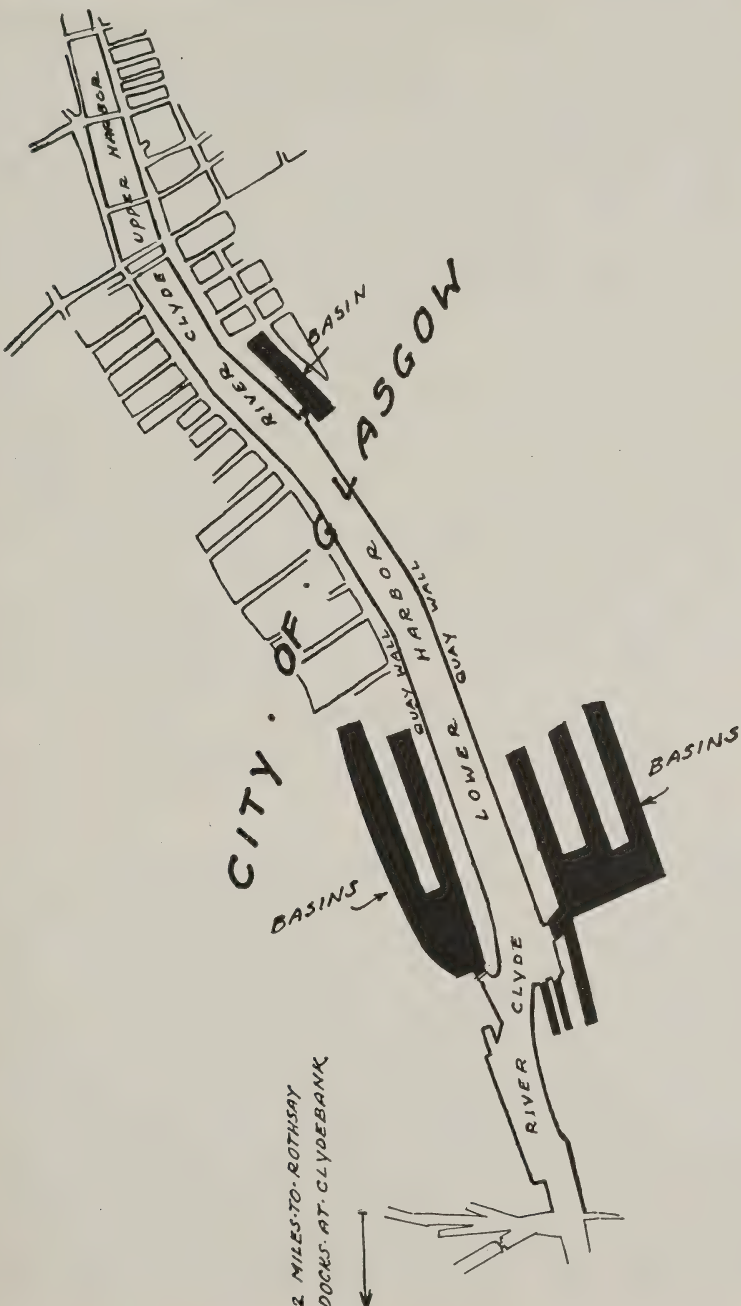
The Cunard and White Star sheds are the most improved and modern in Liverpool. The arrangement of long wide piers in the basins is similar to the pier system in New York, excepting that the piers are built with solid masonry sides, the interior being filled in. They are wide enough for a broad shed close to the quay wall on each side, with a wide paved street between of sufficient width for two railroad tracks in the rear of each shed, with an intervening space or street for truck traffic and storage. About one-half of these sheds are provided with cranes which run along the roof of the sheds. The handling of cargo on the other half is accomplished by the aid of ships' booms and winches.

All of the sheds are very substantially built of brick, with steel rolling doors, practically close to the face of the wall, with no space for railroad tracks or cranes, although the space outside is wider than exists on any of the New York piers, except possibly on the Chelsea Station. Room being apparently plentiful, many large single story enclosed sheds have been built adjacent to, but inshore from the waterfront street, and are used as large distributing yards through which railroad tracks run between platforms, with paved areas for trucks, whither the large amount of cargo which comes to Liverpool proper must be hauled before it is finally placed on railroad cars.

BIRKENHEAD

Across the River Mersey at Birkenhead, about one-third of the total dock and quay room of the Mersey Dock and Harbor Board has been developed.

A large basin has been created in what was formerly a creek, extending from the river, where it is locked off, directly inshore for about 3 miles. Most of this quay front is shedded with brick


UPPER HARBOR
RIVER CLYDE
BASIN
CITY OF GLASGOW
QUAY WALL
HARBOR
QUAY WALL
LOWER
BASINS
BASINS
CLYDE
RIVER
12 MILES TO ROTHSAY
DOCKS AT CLYDEBANK

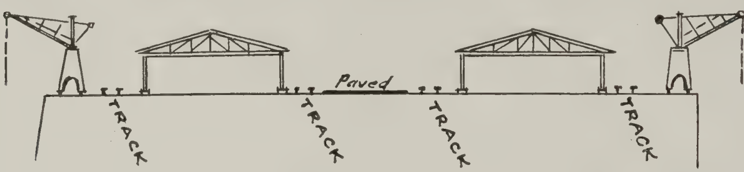
sheds, without cranes, the handling of cargo being done in a manner similar to that in New York, by means of ships' booms and winches.

Here are to be found the large receiving stations and apparatus for the enormous live stock trade. There are many one-story distribution sheds, where railroad cars are loaded, as the larger amount of railroad connection for the Port of Liverpool is made with the dock system at Birkenhead.

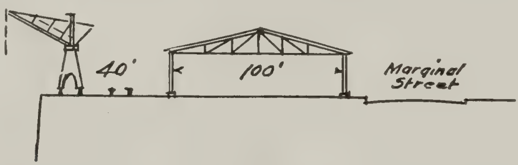
The Board is about to commence the construction of a new dock which will be an extension of the last dock built, extending down the river from Seaforth Battery, as shown on the sketch map, to provide for the newest large steamers now building which are to berth at Liverpool.

GLASGOW.

The harbor waters at Glasgow are under the control of the Clyde Navigation Trustees, which body also has control and jurisdiction



of the river for 18½ miles. The wharfage facilities are provided on both sides of the River Clyde throughout the city, by means



of quay walls constructed of concrete and brick, and also by the creation of unlocked basins. There is a range of tide of about 11 feet, but there the basins have no tide gates. All of the shipping, both on the open river and in the basins, is operated on open tides without gates. The quays are well provided with cranes and, where necessary, with railroad tracks. In the newest basins, wharfage is obtained by extending the quay system out into the basins in the form of piers, wide enough for broad sheds on each side of the pier, with space for railroad tracks and cranes in front, and space for a wide street and storage between sheds (upper sketch).

The Anchor Line is provided with a fine up-to-date steel and brick shed 100 feet wide, built on one of the newly created quays. This shed, while very plain and simple in style, is all that can be desired. It has wooden sliding doors and wooden roofs, but is divided by brick partitions for fire protection (lower sketch).

A peculiarity about Glasgow is the way in which a wide space is left along the face of quay for large cranes and railroad tracks.

The port is well provided with the best class of masonry graving docks. About 5 miles down the river near Clydebank the Clyde trustees have recently constructed the Rothsay Docks, which are used principally for the unloading of ore and the loading of coal, and are capable of accommodating the largest steamers. Enormous towers have been erected, which take the truck bodies of coal cars directly off of the tracks and deposit their contents into the holds of the highest steamers. The whole plant, including power house and appliances, is up to date, and has been constructed on the highest plane of efficiency.

The creation of a port for large steamers and ocean traffic in the city of Glasgow has been accomplished only after gigantic labor on the part of the Clyde Navigation Co., in overcoming the disadvantage of being so far from the sea. This has been accomplished through the expenditure of enormous sums of money in the improvement and control of the river, until now the largest steamers may come to her docks with modern equipment.

In a report of this kind, on account of the short time spent at each harbor, in order to visit all these enumerated in the allotted time, it is impracticable, without undertaking an extensive treatise on each port, to accomplish more than the statement of principal impressions, but I think that the object of the trip abroad has been amply accomplished and conserved by the actual numberless impressions created of the method and means of handling freight and cargo, which experience can not but result with benefit to the city through my department work.

Very respectfully,

CHAS. W. STANIFORD.

Horatio Governeur Wright

Horatio Governeur Wright (see frontispiece) was born at Clinton, Conn., March 6, 1820. He entered the United States Military Academy, July 1, 1837, and was graduated and commissioned a second lieutenant in the Corps of Engineers, July 1, 1841. He served as assistant to the Board of Engineers during 1841 and 1842, and at the Military Academy as assistant teacher of French, and as assistant professor and principal assistant professor of engineering, from 1842 to 1844. From 1844 to 1846 he was assistant to the Board of Engineers, and made a military tour of inspection with the Secretary of War in 1845. From 1846 to 1856 he superintended the construction of Fort Jefferson, Tortugas, Fla., being promoted first lieutenant February 28, 1848. In addition, he had charge during this period of the repairs to the sea wall at St. Augustine, the improvement of the St. Johns River and the Haul-Over Canal in Florida; was made light-house engineer in Florida, and a member of the commission to formulate a project for the improvement of the St. Johns River. In 1854 and 1855 he also acted as superintending engineer in the construction of Fort Taylor and the naval coal depot at Key West, Fla.

He was promoted captain, Corps of Engineers, July 1, 1855, for fourteen years continuous service. From 1856 to 1861 he served as assistant to the Chief Engineer of the Army at Washington, and was a member of the Board on Iron Carriages and Platforms for seacoast guns, and of another board for testing and reporting upon the 15-inch gun as a part of our system of ordnance.

On May 14, 1861, shortly after the outbreak of the Civil War, he was offered a commission as major of the Thirteenth Infantry, but declined it. He was chief engineer of the expedition to destroy the Norfolk Navy Yard, Virgnia, in April of the same year, and served as volunteer aide to General Heintzelman when he crossed the Potomac River and took possession of the heights in Virginia opposite Washington, May 24, 1861. He was in charge of the construction of Fort Ellsworth and other defenses of Washington from May 25 to July 15, 1861, and was chief engineer of General

Heintzelman's division in the Battle of Bull Run, July 21, 1861. From July 24 to September 14, he was chief engineer of an expedition to Port Royal, S. C. He was promoted major, Corps of Engineers, August 6, 1861, and appointed brigadier-general, United States Volunteers, September 14 of the same year.

He commanded his brigade from September 15, 1861, to February 28, 1862, making a reconnaissance of the enemy's works at Port Royal, November 5, 1861, being present at the capture of Hilton Head two days later. From February 28 to June 2, 1862, he was in command of the land forces of the Florida Expedition which captured Fernandina, Jacksonville, and St. Augustine, Fla., and commanded a division in the attack on Secessionville, James Island, S. C., June 16, 1862.

He was promoted major-general, United States Volunteers, July 18, 1862, and commanded the Department of the Ohio from August 19, 1862, until March 26, 1863, and the District of Louisville, Ky., from March 26 to April 26, 1863. He was then placed in command of a division of the Sixth Army Corps and served in the Pennsylvania Campaign, including the Battle of Gettysburg, where he arrived after a forced march of 35 miles. His commission as major-general of Volunteers expired March 12, 1863, but he continued in command of his division and engaged in the Rapidan Campaign from September to December, 1863, commanding the Sixth Corps in the capture of the Confederate works at Rappahannock Station, November 7, 1863. He was given the brevet rank of lieutenant-colonel, U. S. Army, November 8, 1863, for gallant and meritorious services at the Battle of Rappahannock, Va. During the early part of 1864 he was a member of the Board of Engineers to reorganize the system of seacoast fortifications, but returned to command of his division April 24, and on May 9, 1864, he succeeded to the command of the Sixth Army Corps on the death of Major-General Sedgwick. He was given the brevet rank of colonel, U. S. Army, on May 12, 1864, for gallant and meritorious services at the Battle of Spottsylvania. On this same date he was again appointed major-general, United States Volunteers. He participated in all the remaining battles of the Virginia Campaign of 1864, including the defense of Washington against General Early's attack, and the Shenandoah Campaign from August to December, returning thereafter to the operations south of Richmond, where he continued in command of the Sixth Corps until the end of the war.

On March 13, 1865, he was given the brevet rank of brigadier-general, United States Army, for gallant and meritorious services at the Battle of Cold Harbor, Va., and on the same date that of major-general, United States Army, for gallant and meritorious services at the capture of Petersburg, Va. After the surrender of General Lee, he commanded the Sixth Corps in the operations against Gen. J. E. Johnston, and returned with his Corps to Washington, June 2, 1865. During June and July of that year he organized and was in command of a Provisional Army Corps, and from July 20, 1865, to August 18, 1866, he commanded the Department of Texas, and the District of Texas until August 28.

He was mustered out of the Volunteer Service September 1, 1866, and returned to duty with the Corps of Engineers. He was a member of the Board of Engineers and of various other boards until November, 1866, when he became assistant to the Chief of Engineers. He was promoted lieutenant-colonel, Corps of Engineers, November 23, 1865, and continued in charge of various important works under the Corps of Engineers until promoted colonel, March 4, 1879, and brigadier-general and Chief of Engineers, June 30, 1879. He continued in that position until March 6, 1884, when he was retired from active service at the age of 64 years. He died July 2, 1899, at Washington, D. C.

On June 14, 1865, he received the thanks of the legislature of Connecticut for his services during the Civil War, and in June, 1897, the degree of doctor of laws was conferred upon him by Norwich University, Vermont.

Notes on the Theory and Practice of Field Fortification*

MODIFICATIONS DRAWN FROM THE EXPERIENCE OF THE RUSSO-JAPANESE WAR

BY

Mr. F. GOLENKIN
Military Engineer; Lecturer on Fortification in the Nicholas Engineer Academy, St. Petersburg, 1907

This pamphlet contains a series of lectures on the modifications in field fortification which were suggested

1. By the experience of those who took part in the Russo-Japanese War.

2. By articles published recently in Russian and foreign periodicals, and

3. By the proceedings of a special committee, which was assembled under the orders of the inspector-general of engineers.

The following is a summarized translation of a few points in the book:

DEVELOPMENTS OF FIREARMS, ETC.

Recent changes in weapons and equipment must materially affect the question of field defenses. Among modern developments in *weapons* there may be mentioned:

a. The 6.5 mm. and 7.6 mm. improved quick-firing rifles, and their increased range and penetration due to the introduction of the sharp-nosed bullet.

b. Machine guns of the same and of larger calibers.

c. Q. F. artillery: light (guns) of calibers up to 8 cm., and heavier (howitzers) up to 10.5 cm. and 12 cm. (4.2-inch and 4.8-inch), all without recoil and almost all shielded.

d. Heavy artillery (guns, howitzers, and mortars), which may now be considered to belong to the normal armament of provisionally fortified positions. Medium calibers, such as 6-inch and 8-inch artillery, were found to be considerably more effective than heavy, such as 11-inch howitzers, owing to their far greater accuracy.

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e. Hand and mechanically-thrown bombs and grenades, and land mines.

Recent introductions in the form of *equipment* include balloons and kites; field telegraphs (electric and optical) and telephones; field photography; shields, searchlights, and automobiles. The dirigible balloon and its opponent, the automobile equipped with a light gun or machine gun, the combination of aerial kites with photography for the survey of hostile positions, and the proposal of shields for infantry (which has probably been settled by the introduction of the sharp-nosed bullet) have also to be taken into consideration.

GENERAL PRINCIPLES.

Little alteration is to be noticed in the general principle of the fortification of field positions. There are still the *skirmishing line* —the trenches of the front line; the *local reserves*—the closed works of the second line and their attendant batteries; the *general reserve*—the rear position; and sometimes *reconnoitering parties*—advanced posts.

The center of gravity of the defense has now moved forward from the line of works to the front trench line; this is due to the improvement in weapons, which require, for full development of fire effect, the wide extent of trenches, rather than the limited faces of works. The defenders of the trenches must now feel that everything depends on them, and must no longer glance back and think of their own retirement, as they used to do when they felt that the decisive battle would be fought out somewhere in their rear. The line must be reinforced with small works as nucli of resistance in order to give solidity to the trenches. These works will be little more than rings of trench work, surrounded by obstacles, which must not in any way differ in appearance from the rest of the trenches, and must be so sited that they can bring heavy fire to their front, and strong, concentrated, flanking fire over the intervals between them.

The line of works of stronger profile in rear must not be abandoned. If the front line is penetrated by the enemy, this forms a strong retrenchment, from which artillery and rifle fire and counter-attacks can be brought to bear on the front and flanks of the advancing enemy, in order to cause their earlier success to end in their complete destruction.

The ring trenches of the front line must not be replaced by open works or lunettes, which are vulnerable from the rear and easily turned. There is no reason why the closed works should not be quite unrecognizable from the front, when one takes into consideration the numerous subsidiary works, such as communication trenches, blindages, latrines, etc., which lie in rear of a modern defensive line of trenches.

The advantage of *advanced posts* was confirmed at Nanshan, Port Arthur, Liao-Yang and elsewhere. They may fulfil purely

reconnaissance duties, and are then most conveniently placed in advance of the flanks, as being more easy to retire from, and tending to increase the circuit of a hostile turning movement. They should be fortified with open works, and their defenders, though making an obstinate resistance until the enemy arrives within 400 paces, or even later, should not await a bayonet fight.

Advanced posts may also be intended to act as *caponiers* to the main line, to bring flanking fire along its front or to facilitate counter-attacks. In this case, they must be very strongly fortified and obstinately held. It is in rear of such posts and under the concealment of the hills, villages, or woods which are included in them, that so-called ''dagger'' or ''stabbing'' batteries of field or machine guns may be placed to enfilade the approaches to the works of the first line.

Flank reserve positions are prepared in rear of one or both flanks as a protection against turning movements. It is important that these should be in echelon, and not placed obliquely to main line, for the latter does not in any way increase the circuit of a turning movement. With large forces several such positions may be prepared.

Rear positions, intended to cover a retirement, used to consist of lines of strong points. The general opinion of those who took part in the war is that they should now be considerably strengthened and reinforced with rifle trenches and gun pits. Their object is, with a comparatively small garrison, to develop to the utmost the surest means of defense—the strongest possible rifle and artillery fire.

Subsidiary and reserve positions should be occupied by guards, in order to prevent them from being forgotten, as happened at Mukden. In this connection, great importance attaches to notice boards, finger posts and lamp posts, to keep the troops from losing their way.

TECHNICAL REQUIREMENTS OF WORKS.

Passing now to the technical requirements of various works, the experience of the war has shown:

1. That the thickness of earth cover, both in parapets and roofs, must be increased.
2. That the command of all works must be reduced, even down to nothing, if the field of fire permits.
3. The relief of works—that is, the distance from top of parapet to bottom of trench—must be increased.
4. Careful measures should be taken to protect the men against the very destructive, *indirect,* hits of shrapnel and splinters.
5. Concealment from view, and consequently from fire, must be arranged, the enemy's observatories and balloons not being forgotten.
6. Obstacles of all kinds should be used as much as possible.

7. Accuracy of fire at long and short ranges must be provided for, by measuring ranges, setting up distance marks, preparing range tables, and carefully organizing observation stations. The latter may be outside the sections of the position to which they belong and should be provided with observatories and signalling appliances.

8. Lastly, facilities for working at night must be provided, searchlights, star shell, automatically-lighted flares, especially near obstacles, rifles and guns laid in special directions, communications for the reserves, marked by arrangement, easily recognizable in the dark, telephonic and telegraphic communication between sections, etc.

Night attack arrangements include the counter-illumination of the defensive position to paralyze the action of the defense lights; a most careful technical reconnaissance of the ground; the preparation of the most direct approaches and their careful marking, even to the posting of men along them; the provision of means for crossing artificial and natural obstacles, and telephonic communication as in the defense.

It may be taken as a rough rule that, with large forces engaged and provisionally prepared works, the normal thickness of parapets should be calculated to give protection against the shells of 10.5 cm. and 12 cm. field howitzers. With small positions and hasty defenses parapets should be proof against field guns. If it is probable that a position of the latter type is likely to be converted into one of the former, the fact should not be lost sight of in designing the works.

FUNDAMENTAL REQUIREMENTS OF WORKS.

The fundamental requirements of various works are as follows:

1. Rifle trenches. Fields of view and fire as large as possible, allowing no dead ground within 600 to 850 paces. This must be arranged by siting, as clearing the foreground is laborious and unsatisfactory. Choice of profile to suit the comfort of the riflemen. The two service types which allow of "firing standing from a step" and "standing in bottom of trench" are preferable. Some people recommend arm rests, but they are inconvenient for "behind cover" firing, when the rifle rests on the parapet and is held by the left hand under the butt. Secure protection from shrapnel bullets, splinters, and rifle bullets in frontal, reverse, and enfilade fire, and also from side flight of splinters from shells which burst in the trench. The normal interval between men in the trench must be 1½ to 3 paces, and they must be protected with loopholes, overhead cover, *bonnets* (mounds between loopholes) light lean-to roofs, and mined shelters. All kinds of traverses are necessary, recesses for the riflemen are not enough by themselves, as the gangway along the trench must be protected at intervals from enfilade fire. The trenches must be made as invisible as possible, by adapting them skilfully to the contour of the

ground, masking every part of them, and, in provisional fortification, by the use of dummy parapets. All sharp edges and turns are to be avoided, and no subsidiary works should show above the front parapet. Uninterrupted *communication* along the trench, without interfering with the men firing is a necessity, and men passing along this gangway must be defiladed from the view of hostile observers. Communication with flanks and rear is necessary, and if there is no time to dig a communication trench all the way, the safest line should be marked by signposts. If there is time to add a few obstacles in front of the trench, so much the better.

2. "Strong points" or works. Conditions similar to trenches, with addition of an obligatory obstacle, swept by frontal, or better by flanking fire. But parapets must be thicker, blindages stronger and trenches deeper and wider. Safe observation posts for sentries are obligatory. Men spaced at 2-pace intervals. "Refuges,"—that is, trenches placed in rear and to the flanks, connected with the work, for accommodating three-quarters of its garrison during a bombardment, are obligatory. A work should contain living rooms for officers and men, dressing station, telephone office, store room for military stores, and latrine. There should be a supply of water and provisions, also repairing materials, sacks, poles, planks, etc. The drainage is important, and masking more important than with trenches.

Strong points (ring trenches) of the front line and rifle trenches must naturally support each other, the former being thrown slightly forward. If works of stronger profile are required in the front line, they must be open works—lunettes. Even on the flanks they should not be closed works, but should have closed works supporting them in rear. All the works of the second line should be closed works. Trenches between works may be somewhat concave in trace when viewed from the front.

3. Machine-gun emplacements require the same conditions as rifle trenches, and especially an accurate knowledge of ranges. Owing to their mobility, machine guns should be provided with many spare emplacements.

4. Artillery entrenchments require a distant field of fire up to 4,400 or 5,500 yards, and a near field of fire up to the guns themselves in exposed siting. There should be no dead ground. There should be protection for the detachments from all kinds of fire, masking, dummy screens, communications, and obstacles on three sides of the battery. Also in concealed siting the preparation of the foreground, observation stations, referring objects, etc.

5. Cover for reserves requires concealed siting, simplicity of design, convenience of communication with front and rear.

6. Obstacles must be:

a. Large enough in width, height, and extent to form a real impediment.

b. Round closed works they must be continuous.

c. Their distance from the work (40 to 50 paces, and not more than 75 to 100) must not be too great to allow them to be conveniently observed and swept with frontal, and better still with flanking fire; but should be sufficient to safeguard the work from hand grenades thrown from beyond the obstacle.

d. They must be of rapid construction, available materials and simple design.

e. Passages through them must be protected by traverses of the same obstacle.

f. They should be especially designed against the means the enemy is likely to adopt for crossing them.

g. They should be masked either by making them match their surroundings, or by the use of dummy screens.

h. Portable obstacles should be simple, light, and cheap; convenient for packing, fairly reliable for stopping the enemy, and capable of being set up quickly, and even in the immediate presence of the enemy.

FORTIFIED POSITIONS.

The author exemplifies these requirements by producing examples of:

1. The provisional and

2. The hasty defense of an imaginary piece of ground for a small force, consisting of 1 regiment (4 battalions) of infantry, 8 guns, 8 machine guns and 100 cavalry. The extent of front and the depth are each about 2,300 yards, which is about double those of former days, before the improvement in firearms required wider spacing.

In spite of the war experience that fortified positions are turned and are not attacked in front, the study of a position of this small extent is useful training. The position is suitable for the occupation of an advance or a rear guard, and as such it may be first met with by the enemy's advance guard. If it is strong enough to stop this, no turning movement on the part of the enemy will, probably, be possible until his main body begins to arrive. The delay thus caused will give time to the defenders to bring up their main body and to form on this convenient *point d'appui,* or to retire under its protection.

As regards the entrenching of large forces extending for several miles, the great fault of the Russian positions in the war, from first to last, was their continuity—they were uninterrupted, continuous lines of works. The correct method of entrenching a large force, and that long since adopted in Germany, is the line of so-called "tactical or strategic fortified groups," or "rayons."

These rayons are of greater or less extent according to circumstances, approximating to those shown in the examples. They are sited within supporting distance of one another. They require comparatively little labor in their construction, and do not impede

the freedom of maneuvers of the army which rests on their support. The resistance which they offer to an advancing enemy will give the rest of their side time to form on them for defense, or to retire under their protection. But they must be held obstinately, and their defenders must pay no attention to local successes of the enemy in their vicinity. The defense of Sandepu against Grippenburg's attack, in January, 1905, is a good example of the value of such a *point d'appui.*

Occasions may occur when a second and even a third line of groups may be necessary; this is when both sides become involved in a war of positions, as occurred at Mukden, and which a commander should avoid by every means in his power.

Part II deals with details of the construction and siting of the various elements of a fortified position.

ARTILLERY POSITIONS.

Chapter I studies the tactical conditions of artillery positions. The Russians used exposed artillery positions in all the battles down to Telissu, and also at Port Arthur. In the fighting on 16th of May, 1904, near Nanshan, a Russian battery had hardly appeared in an exposed position when it was struck by such a storm of shell fire from Japanese concealed batteries that in a quarter of an hour it lost all its officers and half of its men, without firing a round.

The tactical considerations required in artillery positions are:

1. The greatest possible field of fire both in width and depth; and

2. The fullest cooperation of the fire of artillery with that of infantry, with a view to leaving no dead ground in front of the position.

Artillery positions may be "free," from which fire may be directed through a wide angle of front, or "restricted" to some definite object, such as the enfilading of an approach or the command of some river or ravine.

Concealed positions are only admissible when they satisfy these conditions. Otherwise the guns must be placed close behind the natural crest, for the abuse of concealed positions leads to an enormous expenditure of ammunition with little fire effect. Positions may be 1, open; 2, masked; 3, concealed.

In "*open*" positions, where the guns stand not more than 3½ feet below the crest, there is a large field of fire; direct laying; no necessity for special observers; no cover from the enemy's view and fire; difficulty and danger in occupyng and evacuating the position.

"*Masked*" positions may be defilade: *a.* A man standing (5½ feet). *b.* A man on horseback (8½ feet). *c.* The flash of a gun (14 feet vertically below crest). In *a*, the guns must be rolled up by hand; in *b* and *c*, they can be drawn by horses. In *a* and *b* the flash is visible, in *c* only dust, and the smoke of bursts at the muzzle which, in Manchuria, averaged from 2 to 3 per cent. In *a* the goniometer is not necessary; in *b* it is necessary, but the com-

mander can watch the firing from a platform, and no special point to lay on is required, while *c* requires a specially prepared observing station. In all these positions the guns are discovered as soon as they open fire, and can be destroyed by the system of searching rectangles. This is avoided in "*concealed*" positions where the guns stand at a distance of 400 yards or more in rear of the crest.

Guns may be masked by woods, villages, buildings, bushes, crops, and hayricks, etc., and thus concealed positions may be found on level ground and even on the front slopes of hills. A succession of crests, or of other masks, is extremely favorable. Even concealed emplacements must be thoroughly entrenched, and every form of cover provided for the detachment. Shields only reinforce earth cover. Concealed positions require well-organized observation of fire, accurate passing of order and some technical preparation of the position and the ground in front.

There must be two observation stations for each battery or brigade, one to the front for the observers and the other near the battery for the commander. They may be placed on hills, in which case they require solid and well masked splinter-proof cover, or in trees (but not in isolated ones). Trees which just show over a crest or high buildings, preferably in the middle or back positions of villages, are also suitable. Artificially constructed observatories should be sited so as to admit of observation just over the crest line.

For communication, telephones should always be replaceable by visual signalling. Special signals are better than spelt-out words, being more rapid and less likely to be read by the enemy.

It is also necessary to select subsidiary points to lay on, and to prepare an accurate plan of the whole position. This is the duty of the artillery officers.

The war showed that batteries should be scattered as much as possible, and should be provided with spare emplacements to which the guns may be moved if the enemy discovers the original ones. On the flanks batteries should stand in echelon, to enable them to be swung round if necessary. This may also be necessary in front, where a wide field of fire is desirable.

The difficulty of sweeping with fire the dead ground in front of the guns, caused by the screen over which they are firing, may be met in various ways:

a. By selecting a position commanded by the enemy (the more the ground rises to the front the less dead ground there will be).

b. By drawing back the battery, at a distance behind the crest of the screen.

c. By posting infantry in the dead ground.

d. By combining the positions of batteries in order to sweep one another's dead ground.

e. By placing some of the guns in caponier positions to sweep the dead ground.

f. By combining the posting of guns and infantry to allow of no dead ground. The last two methods are the most practicable.

Infantry standing 700 yards in front of artillery are safe from the effects of premature bursts of shrapnel. In this case the battery must be so sited that, at the least necessary elevation, its shells will clear the crest of the obstacle by twice a man's height. At this same least elevation they should strike the enemy not more than 1,600 yards in front of the defending infantry—that is, the extreme effective range of the latter. Thus the site of the battery may be found experimentally, but the line of sight from it to the enemy's position should always pass 10 to 14 feet below the crest of the obstacle, to give the battery concealment from the enemy's balloons.

Experience showed that guns in concealed positions must cease firing on advancing infantry when it arrives within 600 yards of the defending infantry. It is then intended that two guns out of each 8-gun battery shall be rolled up to a direct position. It is doubtful whether this would be possible, as they would be overwhelmed by the enemy's artillery fire. It is suggested that it would be better to detach some guns from the first for this object, and to put them in carefully concealed positions, where they would lie hidden until required for this special object.

The infantry escorts, at the rate of half a (double) company per battery, should be placed under the orders of the artillery commander, and can be used by him if necessary for replacing casualties among the ammunition numbers, etc. The escorts must entrench themselves where they can best protect the battery; in the case of concealed positions this will probably be on the covering crest, in front of the flanks, and in front-line batteries, to right and left of the emplacements.

The use of dummy batteries, and the careful disguising of actual ones, were fully emphasized during the war on numerous occasions.

The use of "scouting sections" of artillery—that is, the detachment of a few guns into false positions in advance of the flanks, is advised. These positions should be carefully concealed and arrangements should be made for the rapid transference of the guns from one place to another.

Guns of heavier calibers are best sited on the flanks. They require good roads of approach and firm ground, and all the other conditions of ordinary gun positions. In every group of heavy guns it is advisable to add a few machine or field guns for use at close ranges.

COVER FOR GUNS.

The next chapter deals with modern designs for artillery entrenchments. On all occasions entrenching, if not for guns, at any rate for the detachments, is obligatory. The best type is that which provides a separate emplacement for each gun, at intervals of not less than 30 paces. The posting of guns in pairs is not so good, and in battery behind a single bank worst of all. In these latter cases intervals between guns should be from 13 to 17 paces.

Emplacements may be made for: 1. Gun, detachment and wag-

on, or, 2, gun and detachment only, with recesses to take the removable trays containing ammunition. The first should be the normal type, the second only being suitable to the advanced emplacements, with a view to not sacrificing the wagon as well as the gun in case of accident.

When guns are posted at wide intervals it may be necessary, with a view to their concealment from balloon observers, to connect them with a dummy parapet of brushwood or sods, which horses can push their way through if necessary.

The interval between guns in echelon depends on the distance (front to rear) and on the required angle of training, to which must be added half the cone of dispersion of shrapnel bursting at the muzzle. If the required angle is 60 degrees, then,

With distance 15 paces, interval must be 60 paces.
With distance 30 paces, interval must be 90 paces.
With distance 45 paces, interval must be 120 paces.
With distance 60 paces, interval must be 150 paces.

(The above figures are approximate and each includes 12 paces as half the width of emplacements.)

With an angle of 45 degrees the intervals would be 35, 55, 75, 95 paces.

With an angle of 30 degrees the intervals would be 30, 50, 70, 90 paces.

Howitzers require especially large gun platforms—a circle of 8 paces diameter, or a rectangle 8 paces by 4 paces. With mélinite charges they require cover for those making up the charges. Whereas artillery detachment trenches require overhead cover and blindages like rifle trenches, it is not generally advisable to attempt to protect the guns themselves in this way.

It should not be necessary to entrench the wagons of the reserve artillery park or any limbers, natural cover should be available for them in rear of the position.

RIFLE TRENCHES

Trenches sited at the foot of a slope get the advantage of the full range of point-blank fire, but are difficult to connect with the rear. They are allowable when they can make full use of point-blank range, when communication is possible with the rear by means of folds in the ground, diagonally to the front, and when they are not commanded by the enemy. The parapets should be kept low, unless it is necessary to see over bushes or crops in front, or the soil is hard, clayey, rocky, or full of roots.

Great lengths of trench are not advised; they are difficult to apply to the ground and difficult to conceal; if the range of one part is found, the whole is under fire; if the enemy gets into one length anywhere, he spreads along the whole. A half-company trench, 150 to 200 paces in length, is about the limit.

There should be communication trenches to the rear, and in pro-

visional fortification a communication trench parallel to the front behind a reverse slope, which would serve as cover and might be fitted with blindages. The garrisons of the front trenches could use it as a "refuge" during artillery bombardment. The sentries left in the front trenches should have specially strengthened, roofed, lookouts.

Boer rifle pits are not recommended. Supervision is lost, the men are separated and the position only thinly held.

There should be three to four sentry lookouts in each company trench. There must be lookouts at the ends to watch the flanks. The angle of a traverse is a good place for the recess, which must be strongly roofed, with a wide-angle observing slot (60 to 90 degrees) and splinterproof shield at the back. The top of roof must not rise above the parapet. A stool for the observer is a convenience.

Cartridge recesses are easily made by knocking the bottom out of a cartridge box and using the sides for revetting the recess. Allowance for 300 cartridges for each rifleman should be made. By pulling out a sod or two from the revetment of the interior slope, expense recesses for a few cartridges, or food, may be made.

When concealing trenches it should be remembered that observers in balloons may attain the angle of 1 in 4, though more generally they are at 1 in 7, or 1 in 8. The back slopes of trenches facing the front must be disguised accordingly, while trenches at an angle to the front may require to be completely prepared for concealment.

Communication trenches should have the greatest possible relief, and should be as narrow as possible (2½ feet), provided a wounded man can be carried along them. A narrow trench is more quickly made than a wide one, and affords those passing along it better cover. At intervals of 10 to 15 paces, lengths of 4 paces should be widened to double width, to allow stretchers to pass one another. In the intervals between them there should be occasional niches, to allow men to pass.

There should be a large allowance of reserve trenches, and in these all forms of head cover may be omitted, as they will only be occupied in the later stages of the action.

COVER FOR MACHINE GUNS.

The Russians had carriage mountings with shields, as well as pack mountings for their machine guns. Neither of these was quite satisfactory, and a low sledge mounting, which was improvised, was used with success. Sledge mountings are inconspicuous, light, very mobile, can be carried or pulled, or mounted on pack animals or in carts, as desired.

REDOUBTS.

The thickness of parapets in Manchuria amounted to as much as 14 feet, 16 feet, and even 18 feet. The height should not exceed 2.9 to 3.5 feet. The interior trench may be deepened to 7 feet, and

the spare earth may be used as a rear traverse to protect against splinters. Outside ditches are the simplest of obstacles.

Sometimes the ditch is replaced by a belt of entanglement, sometimes the latter is added at a distance of 40 to 60 paces from the ditch as a protection against hand grenades. The Japanese attacks on redoubts in Manchuria were always preceded by a storm of hand grenades.

The most suitable shape of a redoubt is that which conforms best to the shape of the ground, in irregular curves, or broken lines with rounded angles, as flat as possible as a protection against the enemy's artillery fire. A broken gorge can be used for flanking the interval between one redoubt and the next.

Redoubts are nowadays so choked with traverses, blindages, etc., that the bayonet fight can not take place within. The front parapet is the place for this and steps must be arranged for the defenders to climb out to meet the attack. For this purpose stakes are driven in to act as steps, or stakes with wire, or poles, stretched between them. There is no use in preparing the gorge traverse to bring fire to bear on the interior of the work; it must, however, be made high enough to cover the defenders of the gorge from reverse fire.

Communication trenches across the rear of open works should have the rear side sloped off so as to give no cover to the enemy if occupied.

"Whiskers," or wing trenches, gained a certain amount of popularity, partly because they gave some protection to the defenders of the work during a bombardment, and partly because they increased the development of frontal fire. They should not cut through the obstacle zone, and are better placed outside it with no connection with the redoubt except by the rear.

Large traverses must be placed at 12 to 15 pace intervals, and light splinterproof ones at an interval of 6 to 12 paces.

Some authorities say that rear traverses, as a defense against back blast, are as necessary in front faces as in the gorge, but they encumber the work and would give good cover to the enemy in a successful assault.

Observation posts for sentries must be supplied at the rate of not less than one in each face and flank, and possibly in the gorge also.

Blindage roofs, proof against high explosive shells, were made with a double layer of 8¾-inch beams, and not less than 3½ feet of earth. Those proof against the shells of light field howitzers were of two rows of 14-inch beams and 4.7 to 5.9 feet of earth. In order to burst the enemy's shells on the surface they were generally covered with a layer of broken stones or iron plates, the latter being fastened down with pickets.

In works of this description communication trenches should be rather larger than usual, to allow of the more rapid transference of men from the "refuges" into the firing line.

The necessity of having in the work a small store of repairing

materials is strongly urged. A supply of hand grenades, and niches for storing them in, are also required.

Searchlights should not be placed in works, but in the intervals between them. Telephone communication with the rear and with neighboring works should be arranged if possible.

OBSTACLES.

In Manchuria obstacles of wire and explosives were most used. Those of timber and earth were more rarely met with, the first because of the scarcity of timber, and the latter because of the hardness of the frozen ground, and also from their weakness as independent obstacles. Stakes placed checkerwise, palisades, dams with inundations, and shallow military pits were never used.

Passages, of 7 to 25 yards in width, left through continuous lines of obstacles, may be covered by a traverse of the same obstacle placed in rear of the gap, or by letting the two ends overlap for some 12 to 24 yards.

Deep military pits, by themselves, are easily passed and may be used by the enemy to form a "storming parallel," as happened at Liao-Yang. They take very long to make, especially in hard ground, and their one advantage is that material for their construction is always available. Combined with even a weak wire entanglement they form a more useful obstacle. They should be made in not less than four rows, with steep sides and narrow intervals, and if they can be flooded with rain water, with stakes standing in them, they make quite a formidable obstacle.

Wire entanglements are better obstacles, especially with barbed wire. They have the advantage of being very little damaged by artillery fire, even when they are standing fully exposed in the open. The only effective means of making passages through them is by sending up men with wire cutters. If the entanglement is combined with pits, the men can find cover in the latter, but otherwise the Japanese used shields.

The Japanese wire cutters had handles about 4 feet 8 inches long. Captain Modrakh has invented a simple cutter which, fitted on the end of a rifle like a bayonet, can cut both loose and tight wires simply by stabbing at them. This invention has the advantage that the man using it is not disarmed, as he must be when using both hands with cutting pliers. The conclusion of peace before the invention was perfected prevented it from being extensively tested.

The idea of cutting down wire entanglements by machine-gun fire, or covering them with bridges or such like, is impracticable.

A belt of obstacle is better 28 feet than 21 feet, in thickness. It is better not to put the obstacle in a shallow trench with a glacis. But a concealed obstacle, which comes as surprise to the enemy, is of great value. Some concealment is obtained by coating light colored stakes with mud, or grass, or brushwood. They are best placed behind such cover as bushes or crops, which must not, how-

ever, conceal them from the defenders. If the bushes are themselves entangled with wire they form an obstacle very difficult to discover. In less important localities, and against cavalry, this obstacle may be quite low. When material is scarce, two or three rows of wire fence, with the wires not more than 7 inches apart, form a good obstacle.

If the supply of barbed wire is limited, the front row or the top and bottom horizontal lines should be made of it. Even cord has been used in entanglements on emergency.

The Japanese made some use of abattis. Its chief fault is the difficulty of concealment.

Electrically fired land mines and fougasses were extensively used and were very effective against attacks. Solitary mines do not stop an attack; there must be several rows, placed checkerwise, with from 12 to 24 yards between rows.

Automatic mines were also used. "Shrapnel" mines threw bodies like shrapnel shells into the air, where they exploded on reaching the extent of some 10 to 20 feet of wire, by which they were attached to the ground. "Repeating" mines were arranged to fire three charges, the second 3 feet below the first, and so on at intervals of ten minutes. These were intended to destroy men who took shelter in the craters.

The following portable obstacles were extensively used:

Chevaux-de-frise entwined with barbed wire.

Crow's feet chevaux-de-frise, made of three 6-foot stakes, pointed at both ends, and fastened together at the middle, at right angles to one another.

Planks, studded with nails on both sides, either straight or in crosses.

Snares of thick wire, used in connection with the studded planks.

The chevaux-de-frise and crow's feet are useful for making obstacles on ice or frozen ground, or for erecting an obstacle in front of a ditch, in the presence of the enemy. When placed, they must be anchored to pickets and entangled. The nail planks are suitable for exterior slopes of parapets or for the banks of rivers, under thin ice. They were used at Port Arthur.

The Russian hand grenades had wooden handles, and the charge vessel was surrounded by two heavy lead rings. Their range was about 60 paces, while the broken fragments of the rings flew as far as 400 paces. The grenades were fitted with safety catches for convenience of transport.

ENTRENCHING TOOLS.

Tools must be of the best quality, heads of the best steel and helves of strong wood (beech or oak) or hollow steel. The helves should not be too short.

Infantry should carry—besides picks and shovels—axes, saws and wedges, chiefly for use with firewood; augers for use with

trenails, tracing tapes and compasses. Every officer and man should carry something.

Full-sized entrenching tools should be carried in light carts, or on pack animals, immediately in rear of the regiment.

SHARE OF ENGINEERS IN THE CONSTRUCTION OF DEFENSES.

The want of an authoritative statement as to the share required of engineer troops in the fortification of positions has led to many unfortunate misunderstandings: infantry has waited for engineer troops to make cover for them instead of setting to work themselves, with the result of quite unnecessary losses.

The experience of the war has shown how technical troops and appliances should be utilized, and whose duty it is to arrange for their use under various conditions.

In the case of hasty fortification, especially with a small force, the work must be done by the infantry themselves, from their own designs and under their own officers. The commandant of the force, with commanders of units, and artillery and engineer officers (if any) will carry out the reconnaissance of the position, and decide on the spot the general scheme of defense, viz: artillery positions, sites of strong points on flanks and center, general position of main defensive line, reserve and echeloned flank positions, advance posts, rear positions, specially important items in each section and what sections are most important to the scheme of operations.

The decision as to smaller details is the right and the duty of the section commanders. All officers should understand nowadays, therefore, the details of fortification of positions. Only theoretical and practical training on maneuvers can teach the army the peculiar faculty and aptitude for entrenching which was so lacking in the war. Eye-witnesses said that the troops dug well enough, but could not combine their work with tactical operations, the knowledge not only of *digging* but also of *fortifying a position* being possessed by few.

If there are engineers in the force, their rôle is that of skilled assistants, to assist those in charge in their calculations and instructions, in checking the accuracy of the work, and in avoiding mistakes. Engineer officers, if available, will be placed at the disposal of, and be subordinate to, the commanders of sections; they can not interfere with the responsibility of the commandant and the sectional commanders. Their men will be employed as the commandant may decide. They may deal with approaches or communications, or with rear or reserve positions, or with the defenses of specially important strong points.

In the case of provisionally fortified positions, the engineers will draw out the project of defense in all its details and apply it to the ground, laying out all the work (except the rifle trenches, which will be traced by the company commanders) and carrying out much of it themselves. The troops in this case provide working parties,

and their commanders are only responsible for the accurate carrying out of the work allotted them by the engineers.

SPECIAL OCCASIONS FOR THE USE OF FORTIFICATIONS.

In an appendix the author enumerates some special occasions for fortifying field positions.

Positions for the Defense of Defiles. Defiles may be open (river crossings) or concealed (mountain passes). These positions are required either to allow advancing troops to deploy from the march into battle order, or to allow retiring troops to close. In order to protect the mouth of the defile from the enemy's shrapnel fire, the position should be 3 versts (3,600 yards) in front of it.

When the enemy can advance up both banks of a river, the position covering the defile becomes a double *tète du pont.*

A position in rear of an open defile is intended to prevent the enemy from using it. The position should then be within the range of effective rifle and artillery fire from the defile. It is more effective if the obstacle (stream or marsh) takes a curve concave to the direction of the attack.

Positions in rear of concealed defiles are intended to prevent the enemy from debouching from them. The works should be protected against plunging fire.

A position within a defile is intended to obstruct the enemy's advance. It should be placed where the defile widens out or where two or more defiles join. A good réduit, or keep, is necessary to cover retirement, which is difficult.

Points of Disembarkation. These positions are taken up and entrenched by the first troops who get ashore. The flanks may be protected by the guns of the fleet. With large forces there should be several lines, with a strong réduit to facilitate re-embarkation. A position to oppose a landing is fortified like one in rear of an open defile.

Posts on Lines of Communication. These include the base or "initial" station, the "head" station, "intermediate" stations, and sometimes also "concentration" stations, at railhead or where field operations begin. They must be protected from all sides, usually by girdles of detached strong points. A strong réduit, so sited that it will command the interior of all the other works is very important.

Positions covering railway stations, tunnels, and railway bridges are similar to small posts on the lines of communication, blockhouses taking the place of strong points. Communication by telegraph or signal with the neighboring points on the same railway line is important.

Fieldworks in the Attack of Fortified Positions. Now that an attack lasts several days, the attacking troops must entrench in front of the defensive position, especially if their line of advance is obstructed—for instance, by a defile. Such entrenchment will

be used by the holding attack during a turning movement, which may last several days.

Parties of troops who have seized on some important tactical point in the enemy's position will entrench themselves while waiting for reinforcements.

The attacking troops should also prepare a rear position for use against a counter-attack, in the line of their first artillery position. From this they would work forward to the second artillery position, the whole operation being similar to the siege of a fortress.

ENGINEER TROOPS IN THE ATTACK.

Engineer troops in the attack will:

1. Arrange for the passage of natural and artificial obstacles.
2. Prepare approaches to artillery and rifle positions.
3. Put in a state of defense important points captured from the enemy.

The reconnaissance of obstacles, and of the ground nearest to the enemy's works, is the duty of engineer officers. It is carried out by night by special reconnoitering parties, or by day, possibly, from observatories and balloons.

The Failure of the Austin Dam

BY

Capt. AMOS A. FRIES
Corps of Engineers; Member American Society of Civil Engineers

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Austin is located in Potter County, north central Pennsylvania, about 15 miles east of the main line of the Pennsylvania Railroad between Baltimore and Buffalo. A switchback on the Buffalo and Susquehanna Road connects Keating Summit, on the main line of the Pennsylvania, with Austin and other small towns in the valley below Austin. Travelling over these lines the writer visited Austin, November 14, and spent three miserable hours inspecting the dam. The valley is less than 500 feet wide at the site of the dam, and nowhere between it and the town is the width more than about three times as great. Up this valley, on the day of the visit, the wind blew a 35-mile gale with the thermometer below the freezing point and the sun totally obscured by dense clouds.

It is not proposed here to attempt any complete description of the dam, as it is now or as it was prior to the accident, but to call attention to what appear to have been the causes of the failure. The writer had the rare good fortune, through the courtesy of the Engineers Society of Pennsylvania, of hearing Professor McKibben, of Lehigh University, lecture on the construction and failure of the dam. Professor McKibben had inspected the dam three times, the first inspection being two days after the accident, and had made extensive analyses of the stability of the dam under the varying conditions encountered at the dam site. He believes the dam failed through the failure of the foundation, or the bond between the foundation and the dam, and in this belief the writer heartily concurs.

An eye-witness states that a section of the west end of the dam gave way near the bottom first. He states that he saw it start and had time to go into his house and telephone a warning to the town of Austin and get outside in time to see the dam give way.

His evidence is strikingly borne out by the position and condition of the west end of the dam and the broken fragments just below it. About 75 feet of the west end of the dam is intact, the outer end showing a very regular, clean, nearly vertical break about 35 feet in height. The piece adjoining the standing section at this end of the dam lies on its downstream face about 100 feet below the dam, with its top toward the west. The base on which this piece rested is nearly level and fairly smooth. This condition, together with slight evidences of laitance and a number of 1¼-inch twisted steel rods protruding from its upper surface, shows conclusively that there was a horizontal joint at that point. Nevertheless, this

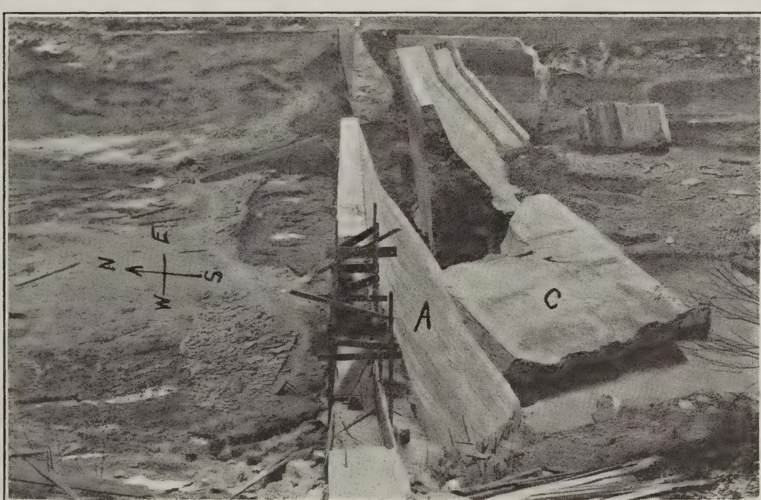

Fig. 1. Austin Dam, November 14, 1911. Looking straight across the dam from the bank 15 feet above the top. C was first piece to go out at west end. B, which was between A and C, lies down stream beyond the limits of the picture. Note broken edge of A in line with top of C, which latter lies with its base down stream; note, also, forms partly in place preparatory to closing gap blasted out in January, 1910, to save the dam.

section undoubtedly did not give way until the next adjoining piece had failed completely, first at the bottom and finally at the top, so that it fell with its top against the base of the piece just mentioned. Calling the three pieces of concrete at the west end of the dam, beginning at the shore, A, B, C, in order, the one just mentioned as giving away first would be C. That this piece gave way first is clearly evidenced by a number of parallel score marks on its present

top surface, exactly in line with the *original* position of B and B's present resting place, 100 feet below the dam. The bottom 10 or 15 feet of the first piece to fail (C) split vertically, so as to leave an irregular wedge-shaped strip standing in its proper place. This strip widens toward the west, and when B gave way it split vertically a large slab from A, the portion still standing. The base of C has adhering to it pieces of the slate of the foundation, indicating clearly that it gave way in the foundation.

The eastern end of the dam broke in somewhat the same way as the western, but only one piece was carried away, though another large one was shifted so as to form a 60-degree angle with its original position.

Both ends must have given away at very nearly the same instant. The 200 feet, more or less, in the center was shifted bodily from 5 to 15 feet downstream, the eastern end being farthest out of line. This section apparently slid on its foundation without rotating at all around a horizontal axis. The rock on which the dam rested is a sandstone, in horizontal layers from 1 to 3 feet thick, with shale and earthy material between—exactly the kind that would fail by sliding.

The concrete seems, generally, to have been good and well made. Many large pieces of the sandstone were put in the concrete and, in every case where visible, they are broken across, showing the stone to have much less strength than the concrete.

The dam, as designed and built, is a gravity section down to the natural surface of the ground, below which both sides were vertical. This vertical downstream face was generally about 4 feet high.

Apparently, it was assumed that the foundation material was strong enough to prevent sliding without carrying the gravity section to the bottom of the concrete. And right there is where the vital mistake in design was made.

Such material as is found at the Austin Dam site should have caused any engineer to take the most extreme care to get deep enough with his full gravity section to make sliding absolutely impossible.

Again, the nature of the sandstone should have shown the designer that the dam would be subjected to more or less upward pressure from seepage water, making a still thicker section necessary.

Not only was the section too light for the foundation on which it was to rest, but that section was not carried deep enough by several

feet to make sliding of the entire mass improbable. Other faults in design and construction are (a) the use of large blocks of weak stone in the mass of concrete, (b) the attempt to make the dam monolithic from end to end without any adequate steel reinforcement to prevent temperature cracks, and (c) in making horizontal joints at various places in the attempted monolith.

But, if the engineer is blamable for a faulty design, what extenuating word is there to be said for the Bayless Pulp and Paper Company, who built, owned, and operated the dam?

As is well known, the dam started to give way in January, 1910, when full for the first time, and only the blasting of holes in the dam saved it. Immediately thereafter the Company called upon the engineer who originally designed the dam, to submit plans for strengthening it, and this he did. A hasty study of these plans for strengthening indicates that had they been carried out the dam would have been safe.

But the Company, utterly regardless of life or property, let the summer of 1910 pass without spending a dollar to strengthen the dam, and in utter defiance of the warning of January, 1910, allowed the reservoir to fill. Result, seventy-seven lives blotted out, over two hundred residences, business houses, and public buildings destroyed, and hundreds of people rendered homeless just as the dismal days of winter were coming on.

To sum up, the failure of the Austin Dam emphasizes again the necessity of (a) building solid concrete gravity dams in vertical sections continuous from foundation to top, (b) of avoiding any upward pressure on the base, and, if that can not be done, to make ample increase in cross section of dam to overcome the upward pressure, (c) to go deep enough to make the foundation secure—no matter what the cost—or give up building the dam.

In pleasing contrast to the reckless disregard of the above requirements in dam design and construction at the Austin Dam, attention is invited to the account in this number of the Memoirs of the care taken to obtain a safe foundation for a dam at Hales Bar in the Tennessee River.

National Rivers and Harbors Congress

BY

Capt. AMOS A. FRIES
Corps of Engineers; Member American Society of Civil Engineers

This Congress met in Washington, D. C., on December 6, 7, and 8, 1911, with a very large attendance, practically every State in the Union being represented.

The Congress was organized eight years ago and reorganized in 1906, since which time Congressman Joseph E. Ransdell, of Louisiana, has been President. It has grown in strength and power for good, keeping constantly as its slogan the phrase "A Waterway Policy, Not a Waterway Project." The men in this organization are working earnestly, and to the very best of their ability for the improvement of our waterways in the best possible manner and at the earliest practicable date, so that the present generation may reap some of the benefits of such improvements. Efforts of this sort deserve the hearty cooperation of every one interested in the greatest factor in civilization—transportation. The resolutions adopted by this Congress, and given below, show more fully the aims and ideals of the organization than any words of mine can.

Resolutions.

The National Rivers and Harbors Congress, representing in its membership every section of the country and every form of business activity, assembled in its Eighth Annual Convention, makes the following declaration of its principles and policies:

a. We urge the adoption by the Government of a broad, liberal, comprehensive, systematic, and continuous policy of waterway improvement, a policy which has been heretofore unanimously pledged by the great political parties of the country in convention assembled, which pledges have not as yet been redeemed.

b. We urge the continuance by Congress of the policy of annual appropriations for rivers and harbors and connected waterways so long advocated by us, and which was adopted by the Sixty-first Congress. We congratulate the people of the United States that the President, their Representatives in the National Legislature and the Executive agencies of the Government are alive to the

nation wide demand for a broad and comprehensive policy of waterway improvement and the sentiment favoring liberal appropriations therefor.

c. We reaffirm our belief in the supreme importance of improved waterways as an essential element in the development of our country, and that such waterway improvements as have been recommended by the Government engineers and approved by Congress should be completed as rapidly as physical conditions will permit. To this end we urge the adoption of the continuing-contract system, wherever practicable, believing that the Government work should be carried on with the same regard for economy, efficiency, and quick results as are required, expected, and demanded in private enterprises.

d. We reiterate our belief that the minimum annual appropriations required to carry forward waterway improvements on a scale commensurate with the importance of the work to be done, is fifty millions of dollars, and call attention to the modest aggregate required for constructive works of permanent and lasting benefit compared with Government expenditures for other purposes, and we further believe the results to be obtained will be of such value to all the people of this country as to justify the issuance of bonds, for works of permanent character, in such sums from time to time as may be required in any year to supplement the amount available from current revenues, to the end that the work of waterway improvement may not be halted.

e. We submit that waterways improved or created by the Federal Government by the use of money contributed by the whole people of the United States should be free for the use of American ships in fair and open competition and on equal terms, without the payment of tolls; but we contend that a water carrier owned, controlled, or operated by a competing land carrier is unfair competition, and in order to preserve to the whole people the benefits of continued, fair competition, so that the beneficent influence of open waterways shall not be nullified by hostile interests, we recommend the enlargement of the powers of the Interstate Commerce Commission, to the end that the Commission may more effectually regulate competing land and water carriers and competing water carriers, and provide for the interchange of traffic.

f. We realize that the full fruition of benefits from improved waterways is dependent upon the provision of adequate terminals, and earnestly urge upon all local interests concerned to at once start the work of providing adequate and properly equipped terminal facilities, so that water borne commerce may be handled with the minimum of cost and labor and the maximum of efficiency and care.

g. For the purpose of encouraging and developing the use of American coastwise and inland waters; compiling and disseminating data and other useful information concerning present and future opportunities for their use, we request that the Congress by suitable legislation provide a bureau presided over by a chief under the Department of Commerce and Labor, or devolve the foregoing duties upon some existing bureau.

River and Harbor Improvements: Progress and Needs in the United States, 1911*

BY

Brig. Gen. WILLIAM H. BIXBY
Chief of Engineers, U. S. Army; Member American Society of Civil Engineers

Mr. President, Members of the National Rivers and Harbors Congress, Ladies and Gentlemen:

Your hearty welcome is again much appreciated by me. When I addressed you last year I felt very doubtful as to the propriety and advisability of so doing, especially as it was a departure from previous precedent; but I have been much gratified during the past year at the favorable reception which was accorded to my address, not only by you at the time it was delivered, but by the public press in its references to the proceedings of your Congress, and by many prominent officials throughout the United States, some interested mainly in water transportation and others in transportation by rail. Your present warm greeting shows me that my last year's action has at least done no harm, and warrants my addressing you again.

It is with much pleasure that, by your invitation, I am here again to add a few words to the discussion of the important problems before you and to assist, if possible, in the determination of your future line of effort.

The work of your association last year undoubtedly aided very decidedly the passage of the river and harbor bill last winter, and has made it much easier for the Federal Congress at its present session to continue its now practically adopted policy of annual appropriations for river and harbor work. I advise, however, that you continue your energetic efforts in the further education of the general public to the advantages of river and harbor improvements, to the desirability of securing appropriations of funds in such amounts and at such times as will insure a business-like

*An address delivered before the National Rivers and Harbors Congress, Washington, D. C., December 7, 1911.

prosecution of such improvement work until it is accepted as just as much a necessity to the whole country as the postal service.

USEFULNESS OF COMMERCIAL ORGANIZATIONS.

I feel in duty bound to urge upon you again this year, as last year, the desirability and even necessity of your giving active, energetic encouragement to the establishment or further development at each large water-front town or city of a live, active, progressive, commercial club or chamber of commerce, or its equivalent, devoted to business objects such as those which concern your association. In such matters your work can best be done by each individual in his own home city. Such commercial organizations should accumulate a solid array of facts, as well as arguments, which can be used by Engineer officers having to report upon the advisability of proposed improvements and be given also by you to your local Congressman for use before the committees of the Federal Congress, when the latter are considering and deciding upon the appropriations to be actually made.

ADVANTAGES OF MEDIUM DRAFT.

So far as inland navigation is concerned, it is to-day far more important that the total mileage of rivers available to moderate-draft steamers drawing from 6 to 9 feet should be increased than that endeavor should be made to secure a deeper draft navigation over short sections of rivers; the deeper draft improvement being worth its cost only near the Great Lake and ocean fronts on sections forming short extensions or connections of other equally deep natural waterways. The utility of railroads in this country was much hampered at the start by the desire of having several different widths of gauge of railroad tracks; but practical experience soon showed the great advantage to be gained by reducing all to a common standard suited to long through routes and allowing the trains to pass from one end of the country to the other without change of cars. The ideal water transportation will not be achieved until after securing such improvements of waterways as will allow the movement of boats of medium draft over all important rivers and connecting canals, and the establishment of junction boat stations and large storage yards or anchorage basins at the mouths of the busy tributaries.

TERMINAL FACILITIES AND THEIR CONTROL.

This year, just as much as last year, one of the features most urgently requiring your careful consideration is the development of improved terminals, and their municipal control, as well as the enactment of such State and Federal legislation as will require all railroad lines to transfer to water lines, and to give through bills-of-lading and the use of all their loading and transfer facilities to any freight which shippers may desire to have consigned over lines of mixed railroad and water routes. There have been published during the past year several pamphlets of great value on the subject of terminal facilities; as those by Professor Clapp, of New York University, on the Navigable Rhine and on the Port of Hamburg, and those of the New York City Dock Commission, and of the Connecticut State River and Harbor Commission, wherein much detail is given as to the best arrangements of docks, etc. Anyone desirous of further examining into the possibilities of harbor terminal development should also consult the reports of a few years ago by the Chicago City Harbor Commission and the Montreal (Canada) Harbor Commission, such reports being on file in many large public libraries and obtainable from the City Commissions concerned. The need of to-day is not in all cases so much the immediate actual construction of extensive terminal facilities as the early preparation for the same, including especially the early acquisition by all water-front municipalities of wharfage fronts, sites, and surroundings, so that when the time arrives for a needed increase of terminal facilities and when people are found who are willing to put up the money for their construction, there will be plenty of free room for carrying such projects to completion, so that all facilities for such work shall be ready, so that all obstacles in the way of such work shall be removed, and so that when such constructions are actually built they shall be available to the general public at moderate rates and free of all monopoly.

WATERWAY IMPROVEMENT ALREADY IN PROGRESS.

Few people realize the amount of river and harbor improvement work already adopted or under consideration by the Federal Congress. To complete the projects already favorably reported by the United States Engineer Department and already adopted by Congress will require the further appropriation and expenditure of over two hundred million dollars. In addition, the Engineer

Department has already recommended about thirty million dollars worth of new work which has not yet been adopted by Congress, and which is therefore as yet entirely without appropriation. The preliminary examination and survey reports ordered by the last two River and Harbor bills will be presented to Congress at its coming session with recommendations for projects (including those for canals along the coast from New England to Texas) which will probably total about ninety million dollars' worth of new additional work, and many of these projects will probably receive favorable action by Congress. All totaled together there are, therefore, nearly three hundred and twenty million dollars' worth of projects already or soon to be before the country. Even at the rate of forty millions per year every year, it will take at least eight years of consecutive annual River and Harbor bills to complete appropriations for these projects already started or called for. During these eight years there will naturally be further demands upon the Federal Government for new examinations and new recommendations. Under such considerations it will become necessary for Congress to make an early decision as to some general outline of policy by which the completion of the work already in hand or in sight can be secured most speedily with the greatest benefit to the country at large. It may be exceedingly difficult for Congress to draw any line between projects that have been reported worthy, upon which it can consistently stand, except perhaps one which will give to all river improvements that are clearly routes of interstate commerce, especially those of through travel between various States, a precedence over other projects where the improvement is within the limits of a single State, or is mainly or largely of local benefit. If the United States shall establish a good standard line of water communication between all the States along the coast and through the Great Lakes and up the interstate rivers, it would seem as if it would be nothing more than fair for each individual State to assume to itself the responsibility of the extension of this general system to all connecting waterways and smaller tributaries so far as they may lie within the individual State. In the same way, on the shore fronts of the oceans and Great Lakes and on the bank fronts of large rivers, it may be found proper or necessary to limit Federal work to the establishment of a safe anchorage and protected harbor of large area, close to and in front of the general shore line, and to require that any extension of this harbor into the interior

shall be either entirely taken charge of by individual States or their municipalities, or that such States or municipalities shall make large contributions to such work as their share of the cost of the general work of transportation improvement within the State.

STORAGE RESERVOIRS, FLOOD PROTECTION, IRRIGATION, BANK PROTECTION, RECLAMATION, DRAINAGE, SEWERAGE.

This year, even more than last, the United States public is waking up to the necessity, or at least the desirability, of conserving all its natural resources and of so directing the improvement of each resource as to obtain the greatest benefit from all other resources connected therewith. While the United States Engineer Department has already recognized the advantages of bank protection to save and protect property, and of levee construction and drainage to reclaim overflowed lands, and of the use of water for irrigation purposes to aid farm production, and of water power development to operate machinery for light, heat, transportation, and manufactures, it has always been obliged to consider that its own special work of river and harbor improvement was restricted to improvements for navigation until expressly otherwise authorized by Congress. Up to to-day, these collateral interests have been directed mainly by the individual States; and whether a Federal control of all such related matters would be legal or even most advantageous to the country as a whole, is still subject to much doubt in Congress as well as amongst the general public.

As a general rule, the most important function of a river is undoubtedly its use as a free, or nearly free, route of transportation, but at the same time the river is also exceedingly useful as a means of water supply for household, municipal, factory, and farm consumption, as a means of dynamic power, and as a means of drainage and sewerage. On the other hand, the river is detrimental and often dangerous as regards its power to destroy riparian properties by erosion, and is furthermore a source of mixed benefit and danger as regards the effects from its overflow.

As a general rule, the availability of the river for irrigation and power is greatest in the upper quarter of its length, where navigation is impracticable. The river is usually most dangerous to property in the the upper quarter and lower half; and its usefulness for drainage, sewerage, or refuse removal is greatest in its lower three-quarters. For direct consumption of its water by

people and factories, quantity and uniformity of flow and purity of water are important features; for irrigation purposes the purity usually becomes non-essential; for power alone the quantity of water, its uniform flow, and height of fall are important. Droughts injure the usefulness of the river for alimentation, irrigation, drainage, and navigation purposes, and have but few, if any, redeeming qualities. Floods, though often causing great damage by bank erosion and by property destruction, are yet often of very great benefit by reason of their fertilizing deposits, which so enrich the river bottom lands that even one good crop in three years will sometimes render the land profitable to the landowner.

The special conditions most favorable to each of the above functions of a river are so divergent that it is usually impossible to establish any river improvement without detriment to one or more of such functions. A reasonable compromise in such matters is all that can be expected; and prominence must be given to the functions most valuable to the locality under consideration.

In Austria-Hungary, for example, the farming industries on the tributaries of the Danube are more important than the navigation interests, and special prominence is there given to drainage all the year, to irrigation in the dry season, and to flood protection in the wet season. On the lower Danube, between Belgrade and the Iron Gates, the navigation interests are preeminent, and the other interests are considerably sacrificed. In Germany and Belgium, the navigation interests seem to predominate; in Holland and England, all interests seem of fairly equal importance; while in France the navigation interests predominate on some rivers, and property interests on others. In none of these countries is there any extensive and general utilization of water power on the navigable portions of rivers; the volume of water flow, combined with the river fall, being in the navigable section too small to allow of the profitable use of the water for power purposes without injury to the interests of navigation. The same variability and conflicts of interests are found in the United States where, as a general rule, the extreme East, North, and West give prominence to water power; the low grounds of the South Atlantic, Gulf, and Mississippi Valley to flood protection and drainage; the high grounds of the Gulf and Mississippi Basin and the great plains to irrigation, while the navigation interests are ordinarily confined to areas not higher than 1,000 feet above sea level, and occasionally to much less heights. Under such circumstances, Federal con-

servation and control of water interests, as a whole, seems difficult and impracticable except within public lands; and State control within State limits, subject only to Federal control of the interests of public navigation, now seems the only immediate and, possibly, final solution of the question. What is most needed at the present time in the United States is a reasonable regulation of all these water interests by whichever party can most readily and effectively exercise control, in such way as to prevent all needless wastes and injuries, to secure early development of everything which can be immediately utilized, and to protect the rest against monopoly until it can be properly developed and used for public benefit.

While storage reservoirs for irrigation purposes, for city and factory use, for navigable canals, or for power on the upper non-navigable portions of rivers, are used to a moderate extent throughout Europe, artificial reservoirs at river headwaters merely to prevent low-water stages in the lower navigable river are not in general or extensive use. Large lakes, like Lake Constance and Lake Geneva, are natural reservoirs and undoubtedly act to regulate the flow of their corresponding rivers; but even in such cases it is probable that the regularity of river flow is more due to the slow gradual melting of the Swiss glaciers above these lakes than to the lakes themselves. For such purposes the Great Lakes from Superior to Ontario are models of perfection, and any further regulation of the St. Lawrence basin by artificial changes in these lakes is likely to cost much more than it will be worth so far as concerns any effect upon the low-water stages.

Storage reservoirs at headwaters of rivers for merely holding back water which might otherwise produce floods are also apparently as yet of no very extensive general use in Europe. In canalized rivers each dam forms a pool which serves as a reservoir in just the place where it can produce its best and fullest effect, and the cost of additional reservoirs at headwaters would rarely be justifiable. The weakest point of an ordinary storage reservoir system for flood prevention is that the most dangerous and injurious floods in a river basin are often produced by heavy rainfall in the middle areas of such basin, while the reservoirs near the headwaters of the river are too high up the river to be of use when most wanted. The disastrous 1909 summer flood in Missouri, Kansas, and Nebraska was a good example of this, and a reservoir system at headwaters would have been sending down water just at the

time when not wanted. The recent floods of the Seine above and at Paris, France, have been due largely to heavy rains over large areas where reservoirs were not practicable. Consequently, in many European countries, such as Austria-Hungary, the protection of property from river overflow is secured usually by levees on each side of the river bank of such height and distance apart that the space between them is sufficient to hold as much water as can fall during several days of heavy rainfall in the basin above, the result of such levees being practically to form a long, narrow, temporary, and intermittent reservoir, requiring several days to fill or to empty, along the full length of the river in the place where most needed, the cost of such intermittent reservoir between levees being no more than the cost of the total of upstream reservoirs that would be necessary to produce an equally useful effect, and the bed of this intermittent reservoir between levees—that is, the occasionally overflowed lands, being useful for valuable farming or grazing purposes in the intervals between floods. While such water control by levees is useless for irrigation or alimentation purposes, yet for reducing to a minimum the property damage from floods, it appears to have proved the most satisfactory solution up to the present time; judging from the statements of various foreign and American engineers as printed in past reports of the International Association of Navigation Congresses at European conventions and from past United States experience in the lower Mississippi River basin.

The diversion of water from rivers by pumping or by canal intakes for irrigation purposes rarely injures navigation at high-water stages, but may often do so at low-water stages by seriously lowering the water levels.

Concerning drainage and sewerage, the present tendency in Europe appears to be toward allowing all reasonably clean drainage water from roofs, roads, and lands to enter freely into rivers, but toward excluding all raw sewage from rivers, so far as practicable, and toward encouraging the development of methods of purifying sewage prior to its deposit in the river and of utilizing portions of it as a low-grade fertilizer, as is done at Worcester, Mass., and some other New England cities. While the free run-off from city and farm drains and from city and country roads and cleared ground is an absolute necessity and does but little harm, except, perhaps, by increasing the height and frequency of floods in the river below,the disadvantages and even dangers from the deposit

of unpurified sewage in rivers subject to use thereafter for drinking purposes are so great as to have already caused prohibitive legislation in many parts of the United States. In the interest of the public health, the more modern methods and prohibitive legislation should either be made compulsory or at least encouraged everywhere in the United States.

Throughout Europe, upon navigable waterways, the river banks below ordinary high-water level are usually well revetted or otherwise protected against erosion, such work being done by the general government as a part of the river improvement. Such work appears, however, to be restricted to what is necessary to secure a proper and well-regulated river channel and bank, and to provide suitable locations for wharves, docks, and terminal facilities, and does not appear to be extended to the mere protection of private properties unless paid for by the beneficiaries. In many cases, especially in France, Germany, and Austria, the general government or any improvement association, by condemnation or otherwise, acquire the riparian properties before commencing or completing the river improvements, by which process the reclaimed lands become sources of profit to the improvement work and help to pay therefor. This practice, so far as legal and practicable, seems worthy of being followed in the United States; and legislation in that direction should be enacted or encouraged for all locations, and especially where the local property owners do not contribute to the river improvement.

WATER POWER DEVELOPMENT.

The water power question has come up so prominently during the past year, not only before many State legislatures, but also before the Federal Congress, and there has been, in many cases, so much apparent misunderstanding of the general situation and so much apparent conflict of statements by the various parties urging, considering, reviewing, or passing judgment thereon, that I take the liberty of urging upon you great caution in accepting any statements which have not been carefully verified, and in advocating any legislation until after much further full discussion by engineers, reliable financial agents, and legislatures. The public in general and even many so-called "technical experts" are often very careless or inexact in both ideas and language in their consideration and discussion of water power questions, and unusual care is necessary at the present time and until legislatures,

Congress, and the courts have arrived at a reasonable agreement as to existing conditions, future needs, and legal possibilities.

Water powers are measured theoretically by a consideration of the volume of water available for use and its head or vertical distance through which it can fall during the generation of power. Practically, however, each water power is usually measured by its developed power, which is such proportion of its theoretical power as can be delivered to the middle man or to the ultimate consumer.

In order that a developed water power shall have any marketable value, there must be a demand for its use within a reasonable distance of the water fall, and the cost of delivery of the power to the ultimate consumer must not exceed the value of the use to which it can be put nor exceed the cost at which equivalent power can be produced from any other material by other processes in that locality. In comparing valuations or rentals of water horsepowers, it is exceedingly important to distinguish between the theoretical horsepower at the crest of the dam, the horsepower when leaving the power house, and the horse power when delivered to the consumer; as it is not at all impossible that the cost in the last case may be as much as one hundred times the cost in the first place. In some cases, water power has value when used intermittently, but in the majority of cases it is commercially useful only when its water supply, or at least its power delivery, continues practically constant and uniform. There are cases on record where it is estimated that the original cost of the acquisition of property and of the construction of water power development works and distribution lines has been one or more hundred dollars per horse power, and that a rental of power to the final consumer has been as high as $75 per horsepower; while on the other hand there are cases where water is still running to waste over natural dams in large volumes, with over a hundred feet of head, because the supply of power to the ultimate consumer is already in excess of his demands and of his ability to use it; and there are some cases where $75 per horsepower delivered to the consumer is fully as fair to both seller and buyer as fifty cents per theoretical horsepower at the crest of the dam. The above explains to a large extent the enormous difference in estimated values of water horsepowers or in the charges made for the rental of water horsepowers in various parts of the United States.

The ownership of water powers on existing streams, while a question of great importance, is still not at all uniform throughout

the various individual States and, perhaps, not yet fully settled in the courts. As a general rule, all ownership of water and of its flow and of the land on the banks and bed of the stream, is vested in the owner of the adjoining land; and his rights, called riparian rights, are almost invariably regulated by the laws of the individual States concerned, except in unusual cases where the Federal Government is still the original owner of the adjoining land, having never ceded it to any individual State. As a general rule, the riparian owner has absolute ownership and control of all land above ordinary high water mark, but only a limited control of the land and bed of the stream below high water mark, and a limited use of the water. As a general rule, he has ownership and control of the water which reaches him and of the water power which may be developed within the limits of his riparian frontage by the use of the water flowing into his property from above while it drops from the level at which it originally entered to that at which it originally left his property; this ownership being subject, however, *first*, to any losses which may come from the reserve by the United States of so much water as is needed for the public interests of navigation, and *next*, to his liability for any damages which he may cause to the riparian owners above him or below him by his use of the water. In some cases, especially in States where water has been considered more valuable for irrigation than for water power, the ownership of the water has been by State laws given to the first users or appropriators of the same instead of to the riparian owners.

Except where the Federal Government is the original owner, as within the forest reserves under charge of the Department of Agriculture, or on other public lands under charge of the Department of the Interior, or where by special acquisition and Act of Congress, as at Sault Ste. Marie, Niagara Falls, and some few special navigable streams under the Secretary of War, the Federal Government has not at present any absolute undisputed ownership of undeveloped water powers. But on all navigable streams, or on those which affect navigation, the Federal Government has a limited control of water and water powers, exercised by the War Department. As a general rule, throughout the United States, as in England, the public right to the use of a river for purposes of navigation to the extent deemed proper by the Federal Government, takes precedence over all other rights; and the use and control of the water and of its flow within the river takes precedence

over all other uses and controls. Where rivers are improved for navigation, the Federal water rights are therefore of two kinds: one that of the public right of navigation—that is, rights for everything necessary to allow free passage of boats through the river—and the other that of original or acquired ownership of riparian properties, which include water and water power and rights of free access to the river at all stages, and of use of all land between high and low water mark when not naturally overflowed by the river. Under existing laws as enforced, whenever the Federal Government undertakes a river improvement it can, free of cost, use so much of the water and water power, and can occupy so much of the land within the river banks, as is necessary in the interest of navigation; but if it wishes to overflow land not ordinarily overflowed by the river itself, or if it desires to occupy the bank of the river outside of ordinary high water lines in such ways as to deprive the owner of his natural use of the same for landings, wharves, or other purposes, it must, in either case, pay damages therefor or acquire the property by purchase or condemnation. In all cases the riparian owner, whether an individual or the State or Federal Government, is considered to own the water power in proportion to his water frontage and to the original fall of the river within the same, and is considered to have the right to the use of water power so far as he can use it without injury to the public rights of navigation, and to the Federal constructions in improvement of navigation, subject to his giving to the United States a reasonable compensation for his use of the Government dam, or to his paying a reasonable share of the cost of securing and maintaining the pool level above the Government dam. Where, therefore, power is developed in navigable waters by the United States as an incident to navigation, the Federal Government always acquires a control of the water power, although it may not always obtain full ownership of the same.

The General Dam Act of June 23, 1910, recognized the fact that the ownership of power developed by dams constructed wholly at private expense is a matter for control by the individual States and not by the Federal Government; the Act directing that a charge be made by the Government only for such part of the total power as results from expenditures by the Government. In accordance with this Act, which must be complied with before riparian owners can build dams in navigable waters of the United States, the United States, through the War Department, is empowered to require the dam owner to furnish to the

United States free of cost such water and such locks, log sluices, fish ways, and other auxiliary constructions as are necessary in the interest of navigation and the fisheries, and the Act reserves to the United States the control of water levels and a right to charge for all benefits received by the dam owner from Government constructions. The Act, which apparently assumes that the dam owner is a riparian owner who has already complied with State laws, would seem to show that there is no objection, in any or all cases, to a cooperation between the riparian owner and the Federal Government in the construction of dams suitable for the joint use of navigation and power development, provided that the cost be divided equitably between the two parties and that the combined development is practicable and not injurious to the public rights of navigation. The Act specially provides that all dams built in accordance therewith shall be so built as to form a part of a comprehensive improvement for navigation. As the riparian owner must comply with requirements of State as well as of Federal laws, it is quite possible that the best cooperation might be achieved by arrangement between the Federal Government and individual States, such as would leave to the latter all questions of private or State ownership of the riparian properties affected and all dealings with the individual riparian owners, as well as the selection of the best methods of utilizing the water to its fullest possibilities and the determination or control of rentals for water used or powers delivered; but the question is still an open one and needs full discussion and a thorough investigation before its legal questions can be satisfactorily settled and the best methods of practical cooperation can be agreed upon. What is most essential in this matter is not so much the present development of the water power as it is such an early action by each State as shall assure the conservation of all potential water powers in such way as to prevent them from being monopolized by private parties during present disuse, and as to make possible at any future day their use to the fullest extent allowable and to the greatest benefit to the general public.

STATE ENGINEER BUREAUS AND STATE CONTROL OR WORK

As levees and drainage are built principally for the reclamation of farm land and of other private properties, and as irrigation systems, including storage reservoirs together with their dams and canals, are built principally for the development of farm property

and the building up of communities, such property and communities existing and being governed mainly by State laws, and as water powers are developed principally for the building up of corporations and business concerns also existing under State incorporation and ordinarily concerned mainly with other developments within a single State, it seems very proper that all these Engineer constructions should be regulated, if at all, by State authority, rather than by Federal authority, and that the Federal Government should intervene only as an advisor or a controller, and should be an executive only so far as such constructions reach within the limits of several States or at least directly affect the development and prosperity of several States. Even in the latter case it would seem most desirable that the activities of the Federal Government should not be directed to work of actual execution of the projects, but should be limited to advice and control, leaving to the local States the actual execution of all local details of construction, operation, and service, as well as the benefits connected therewith.

Because of the present growing probability that the above-named natural resources of land and water must eventually be handled in some such manner as above outlined, it is already urgently necessary that every State of the Union, which has not already done so, should establish at an early date an office of State engineer, or its equivalent, to investigate, report results, advise the State legislature, direct construction operations, and exercise State control of all work of drainage, irrigation, water power construction, and other water utilities within each State, leaving to the Federal Government the control of only such of these constructions as concern such rivers and harbors as do not properly come under control of a single State. Several of the States have already made great progress in such directions. For many years Massachusetts and Rhode Island have had State river and harbor commissions working always within State lines and in harmony with the Federal requirements. New York has had a State engineer directing State canals, State river improvements, and other works of public nature within the State limits. Connecticut has recently organized a river and harbor commission, which has recommended the creation of a State engineer. Illinois has organized an internal waterway improvement commission. Wisconsin and Minnesota have organized water supply bureaus or departments to specially consider water power development and storage reservoirs, and several other States are either doing or preparing to do the same. Now that the

governors of the separate States are assembling once a year in conference to discuss all matters of common interest to the individual States, it will be far easier than in the past to not only organize State engineer offices, but to operate them under common standardized rules and regulations, agreeing so far as possible under their diverse local conditions, and, on the whole, with harmony and without conflict. Any influence you can bring to bear, individually or as a body, to assist in this development of local State engineer offices and local harbor commissions will be of great benefit, not only to the country at large but very directly to the best progress of the work for which your association was specially organized. I am inclined to believe that by the time that such State offices can be properly organized and thoroughly established throughout the country, the waterway movement will have reached such a stage of development as to require all the assistance it can get from each State concerned, as well as what it will naturally receive from the Federal Government. One thing, however, seems quite certain, and that is that when, as has happened in some cases in the past, the local improvement of small intrastate streams can offer early returns of as much as 100 per cent per year for several years to the various individuals and communities most directly affected, and where private control is illegal and unwise and Federal help must be postponed on account of other more important and more pressing interstate developments, it will be good business for the local State to take up the matter vigorously in its own interests even if it has to impose special local taxes or to borrow the money by the best or most available methods.

Book Reviews

The Coming China. *By* Joseph King Goodrich. Published by A. C. McClurg & Co., Chicago. Price, $1.50 net.

To those interested in Eastern affairs and particularly to those who have watched the recent unfolding of the new Chinese national spirit, this book will furnish a welcome source of new ideas. To the military student, as well, it will present a new phase of this problem, which has already been so interesting to him.

The writer has drawn with a clear perspective, a strong but fair picture, which can not but awaken many a speculation as to the future of this wonderful people.

The encroachments upon the Chinese Empire have been enormous, mostly within a comparatively short time. More than sixty per cent of the original territory has been seized by Eastern nations under various pretexts.

Russia has taken Manchuria, Japan has appropriated Southern Manchuria, Korea, and Formosa; France has taken Tonquin; Germany, Kai Chou; and Portugal, Macao.

Her helplessness to resist these encroachments, and the frequent humiliations she has endured from the sneers at her ignorance, stupidity, and obstinacy, have finally driven China to study largely the lessons of defense which those nations have repeatedly forced upon her. She now sees that her rules of conduct, which were suitable to her needs for the last twenty centuries, are no longer sufficient when she must deal with civilized nations. As she learns to bear the responsibilities of government patterned after that of civilized countries, the author thinks she will more and more insist on taking her place among the great powers, for Chinese pride, supported by her latent powers, will finally require a place in the front rank.

Her conservatism and lethargy, which are the result of several thousand years of habit, her commercial independence of outside sources, even of raw materials, her complete satisfaction in the work of her own skilled artisans, in that of her classical writers and of her agriculturists and her arrogant self-assumed superiority, have all been obstacles in the way of cultivating those arts which are now believed by modern civilized countries to be necessary in order to prevent contemptuous encroachment by a stronger power.

Defensive measures are among the above-named arts, for the present helpless plight of China is due primarily to a neglect of those militant measures which many nations find necessary to their very existence. But the author believes a rapid change is now

going on. Sir Robert Hart, in 1900, represented the Chinese as saying: "Why can you not treat us as you treat others? Were you to do so you would find us friendly enough and there would be an end of this everlasting bickering and these constantly recurring wars. Really, you are too short-sighted; you are forcing us to arm in self defense, and you are giving us grudges to pay off instead of benefits to requite."

It will undoubtedly take a long time to efface the resentment caused by the exactions which the more powerful Western people have insisted upon. In the meantime, the changes in Chinese customs and habits are astonishing. The Confucian classics, long thought to be the highest examples of Chinese literature, have been finally abandoned as subjects for the annual examination for governmental positions and more modern subjects chosen. The author thinks this change the most momentous that has occurred for twenty centuries. The next change in importance is the royal decree providing for the adoption of a constitutional form of government to be in actual operation by 1916. Ten years ago there was a single newspaper, of limited circulation. Now there are upwards of three hundred, which are carried by a modern postal system to the farthest parts of the empire. In the autumn of 1905 a great review of troops, drilled and armed in Western style by Yuan Shih Kai, was held in the province of Chihli, to which military attachés of the legations in Pekin went as interested observers. Railway systems have been extended and new arts and crafts are being taught which were unknown a few years ago.

But the most important advance of all is the new education in ways that conform admirably to the standards of Europe and America. Schools and colleges have been established, and the enthusiasm for Western learning was at first so keen as to make it impossible to get enough teachers. The number of students in the various schools and colleges is upwards of one hundred thousand. The author calls this departure of so many millions of people from the precedents of twenty centuries "the most remarkable and decisive educational revolution in the history of mankind." Events since 1894, when Japan went to war with her former teacher, have forced China into a position of suspicion and hatred of her one-time pupil.

The Boxer insurrection in 1900 and its results, the Russo-Japanese war in 1904-1905, in which China was unable to protect herself from both of these contestants and so lost the Loo Choo Islands, Liao Tung Peninsula, and Korea, have all intensified these feelings. When Japan took Russia's place in Southern Manchuria and resolved to stay there, China's hatred for Japan stimulated her to action, and a new determination was born from which these remarkable changes have sprung.

There is some food for thought for American statesmen in the author's following statement:

"Looking for some reason for the tremendous increase in Japan's

armament, a most logical suspicion was aroused; this great army and this huge fleet are to be used against somebody for the purpose of exacting an indemnity sufficient to pay off Japan's foreign indebtedness and relieve her treasury from the inevitable bankruptcy which faces Japan. No adequate provision is being made for paying that debt in an ordinary way; there is no proper sinking fund; each year the budget shows, after juggling with figures has been adjusted and the eyes of the creditors are opened, that there is either an actual deficit or a paltry balance that is not a drop in the bucket as against the hundreds of millions of yen in bonds sold in Europe and America which must some time be paid or repudiated.'' That China believes these preparations are being made with herself as the rich prize in view need not lessen the interest these facts have in this country.

The author's statements are corroborated in the Japan *Chronicle* of April 27, 1911, from which the following is taken:

''In March, 1906, a law was promulgated providing for the creation of special sinking fund to which a sum not less than 110,000,000 yen was to be transferred from the general account every year for the redemption of loans. * * * In 1905, before this heroic scheme was adopted, the national debt amounted to 2,142 millions; in 1910, the total stood 2,650 millions. The national debt was increased instead of being diminished and the value of a sinking fund, when the debt increases at a faster rate than it can be paid off, seems more apparent than real.''

The author is no admirer of Chinese character, and yet he extols their industry, generosity, morality, and intelligence. The evils of opium smoking and the discreditable stand taken by the English, in fastening this vice on the Chinese for the profit there is in the sale of this drug, are commented on by the author at length.

That China will ever become such a powerful and aggressive military nation as to threaten Western civilization the author refuses to believe, but that she will be able before long to defend herself better and take her proper stand among the great powers he thinks may well be accepted. —W. W. H

Gettysburg, the Pivotal Battle of the Civil War. *By* Capt. R. K. Beecham, Army of the Potomac. Published by A. C. McClurg & Co., Chicago.

Truth, candor, and thoughtful criticism, even though the criticism be sharp, in accounts of battles or other stirring events, by those taking part in them, is a decided novelty and most refreshing. Such is Captain Beecham's account of the Battle of Gettysburg. To a first-hand knowledge gained by active participation in the battle, the author has added the fruits of careful study of other writers on Gettysburg, the whole combined and strengthened with his own keen deductions concerning the leaders, and different phases of the battle itself.

The story opens with a brief, but sentimental, introduction contrasting the bitter years of the Civil War with the peace, plenty, and serenity of 1900. Following that the author shows how the South, through the active preparation of years and the natural bent of the people, was far better prepared for war when the first gun was fired at Fort Sumter than was the North. Here the author also speaks bitterly on a subject seldom spoken of, and one to bring the blush of shame to every American. In this he charges that the powder furnished the Union soldiers was far inferior in strength to that furnished by the Confederacy to the men of the South.

Starting in Chapter III with a vivid word picture of the ''Midnight of War * * * For the Army of the Potomac,'' following its bloody repulse from the heights of Fredericksburg, in December, 1862, the author traces rapidly, yet clearly, the movements of the two armies up to the morning of July 1, 1863, when the advance columns met on the banks of Willoughby Run. The tribute he pays at this point to the good common sense of General Reynolds, as shown in the latter's care for his soldiers, is a good index of the author's work throughout the book. Frankly and severely he condemns the pomposity of generals whose first thought was of show and parades. He also touches lightly, but feelingly, upon the inclination of southern writers to minimize the strength of their armies, and then shows with evident justice that at Gettysburg both armies were practically equal in numbers, supplies, and equipment.

Describing the close of the first day's fight, he rebukes with the candid story of his own movements the efforts of many authors to show themselves as having been always cool, collected, and deliberate in battle. He owns to taking a lively part in the wild scramble of the Union troops, on Seminary Ridge, to escape through Gettysburg to Cemetery Hill, where a new line was being formed by Howard, Doubleday, and Schurz at the moment the entire right wing of the Union Army was doubled up and hard pressed by Ewell's corps, which had just arrived from the northeast. Scrambling at the particular time mentioned was sense and not to be ashamed of, and yet how seldom do we find a man who took part in such a scramble willing to admit the fact. At that moment it was to get through the town ahead of the ''Johnnies'' or be captured, but that does not mean that those men were routed. Far from it. Once reformed on Cemetery Hill they were ready for another desperate fight; a fact which Lee himself evidently took into consideration when he sent word to Ewell to take Cemetery Hill that evening, ''if practicable.'' And, right here we see the difference between trained veterans and hastily organized recruits, for once the lines of the latter are broken they are routed; while as soon as veterans get a chance to reform they do so, and are ready for as stern a fight as in the beginning.

A little further on the author rightly declares that the culmina-

tion of Pickett's charge at the Bloody Angle, on July 3, was in no sense the "High Tide of the Confederacy." That tide had turned eighteen hours before when Warren and Hancock, with the Fifth Corps and the immortal First Minnesota, stopped Longstreet's desperate attempt to roll up the Union, left so temptingly held up in the air by Sickle's Salient in the Peach Orchard. After a study of the terrific fighting of that day, one does not wonder that Longstreet demurred when directed by Lee to attack the very center of Meade's intrenched line on July 3.

The following point, emphasized by the the author, is one that every American should remember against probable future wars.

In recruiting their army, the Confederacy adopted the plan of keeping up the strength of the regiments organized at the beginning of the war by constantly sending recruits to fill the places of the men killed, wounded, or captured. In this way raw recruits early became veterans without the terrible losses suffered when complete regiments of green men went into the field for the first time. The latter method was the one followed almost wholly by the Union Army, and it suffered accordingly. It is seldom that one stops to realize that many times whole regiments of the Union Army, even at the time of the Battle of Gettysburg, numbered less than two hundred men.

As an example, the First Minnesota, when it charged a full Confederate brigade on the evening of the 2d of July to hold the gap in the Union lines until reinforcements could be brought up, had only 262 men. The writer has read much of the Battle of Gettysburg, beginning when a lad of twelve or thirteen with Alexander H. Stephen's History of the United States, but no account has appealed to him as has this story by Captain Beecham. It is valuable to every student of the war, whether studying for military purposes or merely as one interested in the great events of our country—A. A. F.

Addresses to Engineering Students. Edited by Messrs. Waddell and Harrington. The Schooley Stationery Company, Kansas City, Mo.

From time to time, it is wise and satisfying to pause and clear the mind of petty thoughts and routine duties and take a general view of the privileges and obligations of our profession.

The work of an engineer is essentially practical, but to succeed in his profession, as in any other, imagination is an absolute necessity unless he is content to follow the beaten track and to remain a follower instead of becoming a leader. To be the latter, an occasional halt is necessary, as it were, to take stock of present conditions and tendencies and to forecast the possibilities and requirements of the future.

The book of addresses that Messrs. Waddell and Harrington have just issued is a collection of word pictures taken from certain eminent engineers who have thus paused and taken a bird's-eye view of engineering life and duties in their broadest and fullest

sense. The addresses are, for the most part, short, and the book can therefore be read in sections with pleasure and profit. An engineer is always a student, and while most of the addresses have been delivered to graduating classes in engineering schools or before engineering societies in various parts of the country, they are so broad in their scope and come from engineers of such known ability, that the collection is of almost equal value to the engineering student of more mature years whose college days are things of a distant past. The profession at large is indebted to the compilers for judgment in selection and labor in editing this book of addresses. —W. D. C.

GUIDE OFFICIEL DE LA NAVIGATION INTERIEURE. Published under direction of the Minister of Public Works of France by Berger-Levrault, editors, Paris, France, 1911. Price, with map separate, 2.50 francs; with map bound in, 3.50 francs; map alone, .75 franc.

This Official Guide of Interior Navigation of the French Republic is a very complete compilation of information relating to their rivers and canals. A series of tables gives complete physical data in regard to the length, depth, locks, bridges, and tunnels, etc., of all the various navigable streams and canals in the State. Following these tables each river and canal is taken up separately, and, in a brief résumé, considerably more information is given than it is practicable to put in tabular form.

The laws governing navigation, and instructions in regard thereto, form the first part of the book. As a hand-book on the conditions of interior navigation in France it is very complete, and fills a vacancy that has been felt materially by one seeking information regarding the present condition of French rivers and canals.

—W. D. C.

Selected Articles of Engineering Interest

Compiled by Henry E. Haferkorn, Librarian, Engineer School.

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering-Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40 Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution
(52) Barge Canal Bulletin.

BREAKWATERS.

Los Angeles harbor. Amos A. Fries. (30), Jan.-Feb., 1912.—Progress of construction of the Panama canal, year ending June 30, 1911. (12), Nov. 8, 1911. I.

CABLEWAYS.

An aerial cableway for handling coal from mine to colliery. B. Benthams. (14), Sept. 28, 1911. I.—High-speed cableways for laying concrete at the Gatun locks. (15), Sept. 16, 1911, Supplement, I; (39), Oct. 1, 1911. I. (14), Sept. 14, 1911. I.—A Spanish cableway of large capacity. (15), Nov. 18, 1911.

CAISSONS.

Caisson sickness and compressed air. L. Hill. (28), Oct. 21, 28, 1911.—Construction of three concrete-lined shafts, for Bunsen Coal Co., near Clinton, Ind. A. F. Allard. (12), Nov. 1, 1911. D. I.—Continuous caisson foundations for high buildings. (15), Sept. 16, 1911.—Foundations of the Bellevue boiler house. (15), Oct. 7, 1911. D. I.—Pneumatic caisson dam foundations, United Fire Companies building. (15), Sept. 16, 1911. D. I.—A pneumatic method of ejecting muck from caissons. (12), Sept. 20, 1911. D.—Reducing timber in caisson foundations. (15), Oct. 7, 1911.

CANALS.

Combined motor-truck and boat service for freight handling. (15), Nov. 4, 1911.—Distribution system of Pueblo-Rocky Ford irrigation project. (15), Nov. 18, 1911. D. I.—Economic canal location in uniform countries. Discussion. H. J. Doolittle, and others. (21), Oct., 1911.—Feasibility of electric operation on the Morris canal. (15), Nov. 25, 1911.—Methods and costs of construction on contract section 11, N. Y. State barge canal. E. J. Beck. (12), Oct. 18, 1911. D. I.—Method and cost of repairing large canal break. E. M. Chandler. (12), Nov. 8, 1911. I. New York State barge canal. W. B. Landreth. (45), Sept., 1911. D. I.—Progress of N. Y. State barge canal. N. E. Whitford. (28), Sept. 23, 1911. I.—Sewage dilution in the Chicago drainage canal. (15), Nov. 11, 1911.—Problem of the main drainage canal at Chicago. (15), Nov. 11, 1911. D.

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Cement lining for steel pipes on Catskill aqueduct. New York City water supply. (14) Nov. 2, 1911. D. I.—A mechanical shaker for fineness tests of cement. A. D. Gates. (14), Oct. 19, 1911. D.—Some of the properties of oil-mixed Portland cement mortar and concrete. L. W. Page. (21), Sept., 1911. D. I. Discussion. S. D. Newton, and others. (21), Nov., 1911.

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The report of the Royal Commission on Coast Erosion. W. Pitt. (32), Nov., 1911.

COAST PROTECTION.

De werken ot verdediging van de Noordzeekust tusschen de Heldersche en Pettemer zeerweringen. A. T. DeGroot. (16), Sept. 2, 1911. I.

COFFER-DAMS.

The foundations of the Bamberger building, Newark. (15), Oct. 14, 1911. D. I.—A large cofferdam built with steel sheet piling. (14), Sep. 21, 1911. D. I.—Progress at Keokuk. L. H. G. Bousearen. (Stone & Webster public service journal), Nov., 1911. I.—Water power development on the Mississippi River at Keokuk, Iowa. (14), Sept. 28, 1911. D. I.—Treatment of the foundations of the power house and dam at Hales Bar, Tenn. C. H. Tisdale. (30), Jan.-Feb., 1912. D. I.

COLORADO RIVER.

Control of the Lower Colorado River: What methods should be adopted? (14), Dec. 7, 1911.—How the U. S. spent a million dollars in an ineffectual attempt to control the Lower Colorado River. H. T. Cory. (14), Dec. 7, 1911. D. I.—Reply to Mr. Cory's article on the Colorado River break in 1911. J. A. Ockerson. (14), Dec. 7, 1911. I.

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Alkali as a source of danger to concrete dams. Letter to ed., E. T. Tannatt. (15), Dec. 2, 1911. I.—Concrete casings filled with sand as wooden pile protection. T. Englehart. (14), Oct. 5, 1911. I.—Concrete dam across the Mississippi. (40), Oct., 1911. D. I.—Concrete dam at Marquette, Mich. (39), Sept. 1, 1911.—Concrete drop shafts in shifting sands. P. F. McDonald. (40), Sept., 1911. D.—Concrete scows on the Welland canal. (7), Sept., 1911. I.—Concrete work in the locks. (41), Oct. 11, 1911.—Condition of concrete structures in Boston harbor. S. C. Willis. (15), Sept. 23, 1911.—Construction of three concrete-lined shafts, for Bunsen Coal Co., near Clinton, Ind. A. F. Allard. (12), Nov. 1, 1911. D. I.—The contractor's view of city contracts and specifications. C. A. Crane. (14), Nov. 23, 1911.—Constructing a concrete headgate in winter. J. D. McGaughey. (15),Oct. 14, 1911 D. I.—Demolishing a reinforced-concrete grandstand with a pile-driver ram. C. M. Stegner. (14), Nov. 23, 1911.

I.—Design of eccentrically loaded concrete members reinforced on one face only. Discussion. C. W. Martin, and others. (45), Oct., 1911. D.—Discharge-increasing lip for the blow-off pipe of a dam. D. F. McLeod. (14), Oct. 19, 1911. D. I.—Economical steel clamp for concrete column forms. (12), Nov. 1, 1911. D.—Effect of electric currents on reinforced concrete. (15), Nov. 25, 1911.—Failure of a reinforced-concrete reservoir. E. M. De Burgh. (Minutes of proc., Institution of Civil Engrs., London), v. 180, D.—Failure of the Austin dam. A. A. Fries. (30), Jan.-Feb., 1912. I.—High-head hydroelectric development in New York. (15), Nov. 25, 1911. D. I.—A high rock fill dam with concrete facing in Colorado. (15), Nov. 4, 1911. D. I.—Methods and cost of constructing a rock-fill dam with concrete face in Australia. A. J. Debenham. (12), Oct. 25, 1911.—Methods and costs of construction on constract section 11, New York State's barge canal. E. J. Beck. (12), Oct. 18, 1911. D. I.—Morris dam of the Waterbury water works. H. G. Payrow. (15), Nov. 25, 1911. D. I.—Movable forms for a concrete-paved slope. (15), Dec. 9, 1911. I.—Patching holes in a concrete sea wall with the cement gun. (15), Oct. 21, 1911. I.—Provision for openings in reinforced-concrete slabs. E. Godfrey. (14), Oct. 19, 1911. I.—Reinforced concrete bridge across the Almendares River, Havana, Cuba. E. Klapp and W. J. Douglas. (21), Sept., 1911. D. I.—Reinforced concrete raft foundations for tall buildings. (15), Nov. 25, 1911. D.—Reinforced concrete reservoirs at Somerset. L. E. Chapin. (15), Oct. 28, 1911. D.—Reinforced concrete standpipe. W. W. Clifford. (21), Sept., 1911. D. I.—Safety factors in waterproofing. M. H. Lewis. (40), Nov., 1911. D.—Some of the properties of oil-mixed Portland cement mortar and concrete. L. W. Page. (14), Oct. 12, 1911. D.; (21), Sept., 1911. D. I.; Discussion. S. D Newton, and others. (21), Nov., 1911.—Study of a design for a bridge of reinforced concrete arches of unequal span with data on arch settlement. E. Klapp and W. J. Douglas. (12), Nov. 29, 1911. D.—Tests of reinforced concrete telegraph poles. (11), Oct. 20, 1911.

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Conveyor system for handling coal. H. J. Edsall. (49), Dec. 2, 1911. D. Steamers and fuel barges equipped with conveying apparatus for discharging cargo. (14), Oct. 14, 1911. D. I.—Transporter bridge over the River Tees. (10), Sept. 29, 1911. D. I.

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Crane specifications. E. G. Fiegehen. (11), Oct. 20, 1911.—Les installations mecaniques du port de Rotterdam. W. Cool. (16), Oct. 21, 1911. D. I.—Locomotive crane equipped with an attachment for pile-driving. (12), Nov. 22, 1911. I.—Water power development on the Mississippi River at Keokuk, Ia. (14), Sept. 28, 1911. D. I.—30-CWT. Electric "Barry" single-beam transporter. (11), Sept. 22, 1911. D. I.

DAMS. (See also Concrete, Floods, Foundations, Reservoirs.)

Alkali as a source of danger to concrete dams. Letter to ed., E. T. Tannett. (15), Dec. 2, 1911. I.—Another account of the Austin dam failure. (15), Oct. 14, 1911. I.—Another Austin dam failure and its lessons. (14), Oct. 5, 1911.—Another undermined masonry dam. (15), Nov. 11, 1911.—The Austin dam disaster. (10), Oct. 6, 1911. D. I.—Austin dam failure. W. E. Belcher. (15), Dec. 9, 1911.—Austin dam failure. W. H. Sawyer (15), Nov. 4, 1911.—Building the world's highest dams. C. J. Blanchard (American Forestry), Dec., 1911. I.—Concerning instruction in hydraulic engineering; ice pressure against dams. R. Fletcher. (14), Oct. 26, 1911.—Concrete dam across the Mississippi. (40), Oct., 1911. D. I.—Concrete dam at Marquette, Mich. (39), Sept. 1, 1911.—Construction of the Morena rock-fill dam, San Diego County, Cal. M. M. Shaughnessy. (21), Oct., 1911. D. I.—Curved or straight dams; is dam design a matter of mystery? (14), Oct. 26, 1911.—Description of the broken dam at Austin. F. E. Schmidt. (14), Oct. 5, 1911. D. I.—Design and construction of the Morena rock-fill dam, Cal. L. W. Page. (12), Nov. 15, 1911. D.—Design of dams. (15), Nov. 18, 1911.—Design of dams. W. H. Richards. (15),

Nov. 25, 1911.—Design of earth dams. C. T. Johnston. (15), Sept. 23, 1911. D.—Design of masonry dams. E. Wegmann. (14), Nov. 16, 1911. D.—Destruction of the Austin dam. (10), Oct. 13, 1911. I; (15), Oct. 7, 14, 1911. D. I.—Destruction of two earth dams by over topping; flood at Black River Falls, Wis. (14), Oct. 12, 1911.—Discharge-increasing lip for the blow-off pipe of a dam. D. F. McLeod. (14), Oct. 19, 1911. D. I.—Expert testimony at the Austin dam investigation. (15), Nov. 11, 1911.—Extracts from the report of the Isthmian canal commission for the year ending June 30, 1911. (14), Nov. 9, 1911. I.—Failure and repair of a low masonry dam. (15), Nov. 11, 1911. D. I.—Failure of a concrete dam at Austin, Pa., on Sept. 30, 1911. (14), Oct. 5, 1911. D. I.; (27), Oct. 14, 1911. I.—The same. A. A. Fries. (30), Jan.-Feb., 1912. I.—Failure of the Dells and Hatfield dams and the devastation of Black River Falls, Wis., Oct. 6, 1911. (14), Oct. 19, 1911. D. I.—Flood prevention and protection at Pittsburg. (12), Nov. 15, 1911.—The Halligan dam. G. N. Houston. (21), Oct., 1911. D. I.—High-head hydroelectric development in New York. (15), Nov. 25, 1911. D. I.—A high rock-fill dam with concrete facing in Colorado. (15), Nov. 4, 1911. D. I.—Hydroelectric development on the Watauga River. (15), Nov. 11, 1911. D. I.—Hydrostatic pressure beneath dams. Letter to ed. G. M. Brauner. E. Godfrey. (14), Nov. 30, 1911.—Ignorance of the coefficient of sliding friction between rock substrata a cause of dam failure. (12), Nov. 1, 1911.—Masonry dams and their foundations. F. P. Stearns, A. D. Flinn, E. Wegmann. (14), Oct. 21, 1911.—Methods and cost of constructing East Park dam, Cal., Orland project, U. S. Reclamation Service. E. G. Hopson. (12), Oct. 18, 1911. D. I.—Morris dam of the Waterbury water works. H. G. Payrow. (15), Nov. 25, 1911. D. I.—Movable dams for spillways in connection with earth dikes. P. M. La Bach.—Opportunities for contractors on U. S. improvements on the Upper Mississippi River, with description of plant required and data for estimating. C. W. Durham. (12), Oct. 18, 1911. D. I.—The partial failure of a concrete dam at Austin, Pa., on Jan. 23, 1910. (14), Oct. 5, 1911. D.—Plans for the completion of the Gatun dam spillway. (14), Nov. 9, 1911.—Plant and construction lay-out on Keokuk dam. (39), Nov. 1, 1911. D. I.—Progress and cost of construction of the Panama canal. (12), Nov. 8, 1911. I.—Progress of N. Y. State barge canal. N. E. Whitford. (28), Sept. 23, 1911. I.—Provision for uplift and ice pressure in designing masonry dams. C. L. Harrison. (14), Nov. 30, 1911; (21), Nov., 1911.—Reservoir breaks on the Black River, Wis. (15), Oct. 21, 1911. D. I.—The San Luis Rey River dam. (39), Sept. 1, 1911.—Some thoughts suggested by the Austin dam failures. J. R. Freeman. (14), Oct. 19, 1911.—Treatment of the foundations of the power house and dam at Hales Bar, Tenn. C. H. Tisdale. (30), Jan.-Feb., 1912. D. I.—Upward pressure under dams. E. Godfrey. (15), Nov. 4, 1911.—Water power development on the Mississippi River at Keokuk, Ia. (14), Sept. 28, 1911. D. I.—Should the factor of safety of dams be fixed by law? G. E. Ladshaw. (14), Oct. 19, 1911.—Should dam construction be under Federal control? D. M. Andrews. (14), Oct. 19, 1911.

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Dock and wharf equipment. J. L. L. Holmes. (70), vol. 180, D.—Physical characteristerics of European harbors. C. W. Staniford. (30), Jan.-Feb., 1912.

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Demolishing a reinforced-concrete grandstand with a pile-driver ram. C. M. Stegner. (14), Nov. 23, 1911. I.

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Construction of the Morena rock-fill dam, San Diego County, Cal. M. M. Shaughnessy. (21), Oct., 1911. D. I.—A new theory for the design of reinforced concrete reservoirs. Discussion. (45), Sept., 1911.—Failure of a reinforced concrete reservoir. E. M. De Burgh. (70), vol. 180. D.—Flood damage at the Hatfield reservoir, Wis. (15), Oct. 14, 1911. D. I.—Sluice gates in the rock tunnel of the San Luis Valley irrigation district, Colo. (14), Oct. 5, 1911. I.—River regulation by reservoirs. (15), Nov. 11, 1911.—River and Harbor Improvements: Progress and Needs in the United States, 1911. Gen. Wm. H. Bixby. (30), Jan.-Feb., 1912.

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Editorial Notes

The Memoirs, Bi-monthly

Three years ago the first number of the MEMOIRS was issued after much hard, discouraging work by those who planned and begun it. To get officers and others interested in the proposed publication and get them to subscribe $5.00 each in advance; to collect suitable articles; and, more than all, to get advertising for a magazine which was admittedly an experiment, must many times have seemed hopeless.

Yet, on the foundation then laid the MEMOIRS has endured, prospered and grown. Even at that time it was planned to make the magazine a bi-monthly one just as soon as its finances warranted it. That time has now arrived and it gives the editors great pleasure to announce that, beginning with this issue, the MEMOIRS will be published every two months. The subscription price will remain the same and every endeavor will be made to maintain the present high standard of the magazine.

We have now four hundred subscribers, of whom fifty are libraries and colleges, showing the appreciation of the magazine outside the Engineer Department of the Army. The number of subscribers can be very largely increased if each one interested in the MEMOIRS will help just a little. This we hope you will do, remembering at the same time that good articles are the best advertisement, and that you can help us by writing up the work you are engaged upon.

To all younger officers we would say that the ability to write well is one of the things that all engineers should be able to do, and that this ability comes only with practice.

To Advertisers

Beginning with this number, the MEMOIRS will be issued every two months, making six issues per year instead of four. The yearly rates for advertising will remain the same as heretofore. That means that each advertiser gets a 50 per cent increase of publicity without increase of cost.

In reality, it means a good deal more than 50 per cent increase,

because with this change in the number of issues there is begun a systematic effort to increase the subscription list.

Heretofore, no such effort has been made, and yet there are more subscribers to-day—including fifty colleges and libraries—outside the Corps of Engineers than in the Corps itself.

No profit is made on the magazine, which accounts for the fact that as the subscription list and number of advertisers has increased the editors have been able to decrease the subscription price from $5.00 to $3.00, and to increase the number of issues from four to six yearly without increase in advertising rates.

Public Water Terminals

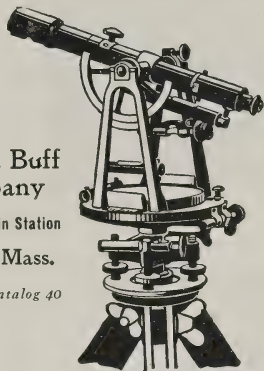
One of the most vital and most urgent necessities needed to revive and stimulate water transportation is public ownership and operation of wharves, warehouses, railroad terminals, and storage yards on every harbor and at every river landing.

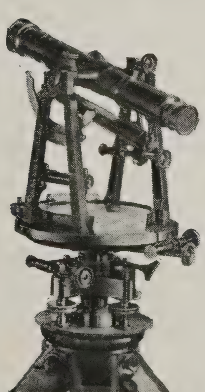
So long as there was railroad competition, the lack of public water terminals was not greatly felt, but to-day when no such thing as railroad competition exists, and when even the majority of steamship lines are controlled by the railroads, the only possible hope, short of public ownership and operation of all transportation, is public ownership and operation of water terminals. These will give the independent man with money enough to build a boat or two a chance to go into the transportation business. The amount of capital required to start such a business will depend on whether it is to be carried on over ocean routes or along inland waterways.

Of course, it is a question whether public water terminals even will revive water transportation, but if it does not and the other alternative of public ownership of all transportation becomes necessary, public water terminals are a step in the right direction, as they are needed in either case.

It is thus that to-day we see so many cities in this country following the lead of New York and San Francisco, in acquiring the ownership and operation of their water front facilities. Notable examples of efforts in this direction are seen in Chicago, Boston, San Diego, Cal., Los Angeles, Oakland, Portland, Oreg., and Seattle.

San Francisco to-day leads them all in public water front facilities and yet how very far even San Francisco is behind Antwerp, Liverpool, Hamburg, and a score of other European harbors.





The modern European harbor, such as Antwerp, is one of wide slips with still wider tongues of land between which are found not only temporary warehouses, but permanent ones also and, in addition, room for wagon roads, railroads, and even manufacturing plants. Those American harbors that are yet only partially developed and able to plan their harbors along the best lines will have an enormous advantage over those old established ones where great business blocks come close to the water's edge and where the only storage facilities are those which can be supplied on narrow, open work piers, that are not only expensive in first cost but expensive to maintain and operate, for it must not be forgotten that the mere possession and operation of public water terminals will not invite or stimulate the growth of transportation. To do so, those facilities must not only be ample, but they must be so arranged that vessels may be loaded and discharged in the shortest time possible and at the most economical rate. These requirements necessitate broad piers, preferably solid fills, equipped with the most efficient and up-to-date labor-saving devices, such as travelling cranes, overhead trolleys, etc. These facts must be borne carefully in mind whenever the public takes up the acquirement or extension of harbor facilities or else the charges necessary to maintain and operate them will be so great as to handicap that harbor in relation to other harbors, especially European, unless indeed the citizens are willing to tax themselves in order to keep the harbor going.

The primary reason for public ownership and operation of water terminals is that private parties will not build and operate these facilities except at a profit, and even then only when congestion forces it, while the public can well afford to build and operate them at cost on account of the business and wealth the growth of commerce brings to the city and country immediately adjacent to the harbor.

That the mere possession of public facilities does not make low port charges is well exemplified in the case of New York City, which is notoriously one of the most expensive ports in the world for handling ocean business, notwithstanding its great extent of publicly owned water frontage. The cause of this condition is not public ownership and operation, but the lack of space for proper warehouse facilities, storage yards, and, above all, a belt line terminal road connecting with all railroads reaching the city.

That New York realizes this condition is evident from the effort she is making, as exemplified by the article by Mr. Staniford in this number, to learn the best labor-saving devices and the best means of installing them on her present wharves or on others to be hereafter constructed, including the proposed extensive improvement of Jamaica Bay, Long Island.—A. A. F.

An Engineers Dinner

In the last number of the MEMOIRS the editor calls attention to a conference of the district engineers of the district, held in Nashville in the spring of 1911, and recommends a similar conference in the various engineer districts throughout the country. It might be interesting in this connection to tell of an engineer dinner held in this district last winter and one held the winter before. It is expected to have another one some time this winter.

The dinner was not limited to the engineers of the district, but included all who had the more responsible positions in the field as well as practically all the men in the main office. Besides the military assistants and the assistant and junior engineers, a number of overseers, surveyors, superintendents, masters, foreman-carpenters, etc., came in from the field. There were about forty men at each of the dinners.

Dinner was had early in the evening, followed by speech-making, the toastmaster calling on a large number of the men for remarks concerning work and conditions in their parts of the district, or other observations they saw fit to make. Several of the men had notes for their talks, and in one case a man wrote out his entire speech. The speech-making and the dinner generally was apparently of a much more informal nature than that at the Nashville conference, but it gave the men an opportunity to exchange views and get acquainted, and all present seemed to think it was both a pleasure and a benefit.—F. W. A., WHEELING, W. VA.

Prizes for Volumes 3 and 4

On page 164 of No. 9, of the PROFESSIONAL MEMOIRS, announcement was made that one year's free subscription would be given each civilian assistant of the Corps of Engineers, including both engineering and clerical assistants, who should publish in any volume an article of five thousand words—twelve pages or more.

In addition, a prize of $25 was offered for the best article published in each succeeding year by the same authors.

In accordance with the above offer, free subscriptions have been awarded for articles printed to date and the authors notified accordingly, while a board of assistant engineers has been appointed to decide on the best article published in Volume III. The members of the board are Mr. W. M. Gardner, Memphis, Tenn.; Mr. H. C. Gould, Washington, D. C.; and Mr. Edmund Moeser, Zanesville, Ohio.

The Board of Editors desires to express its thanks to these men for agreeing to serve on this board, as it involves a considerable amount of work with only the sense of duty well done for recompense. While not so stated in the original announcement, articles of any length whatever are to be considered in awarding the $25 prize.

Recognizing that the preparation of a good article takes time and study, and the time having arrived when the finances of the MEMOIRS warrants it, the editors have decided to offer four prizes for the four best articles published in the six numbers of Volume IV by engineering or clerical assistants of the Corps of Engineers. The prizes will be $50, $25, $15, and $10, and all articles of whatever length published by the above-named assistants will be considered in making the awards.

Professional excellence, clarity, and conciseness are considered of prime importance, though literary excellence will be given a certain value in considering the awards.

The awards will be made by a board of three non-competing assistant engineers, who will be asked to serve after the last number of the volume is published.

As in the past year, one year's free subscription will be given to each civilian assistant who publishes during the year an article of twelve pages or more.

Mr. Caryl D. Haskins

It is with the regret that one feels over the loss of a dear and valued friend that the MEMOIRS announces to its readers the death of Mr. Caryl D. Haskins on November 18, 1911, at Salt Lake City, Utah, at the age of forty-four.

Mr. Haskins had long been a friend of the Engineer School, as well as of the whole Army, which accounts for the fact that the very first article printed in the MEMOIRS was his lecture before

the Engineer School on ''Light and Power Switchboards,'' on which subject he was an authority, having been at the head of the lighting department of the General Electric Company. Two years later, in Volume III, Mr. Haskins contributed another article on ''A Civilian's Suggestions for a Technical Reserve for the Army.'' This article, which attracted a great deal of attention, shows the broad-mindedness, patriotism, and versatility of the man.

Always taking a keen interest in life, and full of the joy of living, always pleasant and thoughtful of those around him, and always willing to do more than his share of whatever work fell to his lot, Mr. Haskins was the type of man all too rare and one whose place it will be difficult to fill.

Errata, No. 12

The following errors in printing occurred in the No. 12 issue: Page 572, seventh line from the bottom, ''a fall of not more than 8 feet,'' should read ''a fall of not more than .8 foot.'' Page 574, eleventh line from the top, the words ''lateral canal,'' should read ''lateral canals.'' Page 598, fourth line from the bottom, ''Plate III'' should read ''Fig. 1.''

Wanted

Copies, in good condition, of No. 2, Vol. I, of the Memoirs, for which the publishers will pay $1.25 each, either as credit on subscriptions or in cash.

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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VOL. IV. | MARCH-APRIL, 1912. | No. 14

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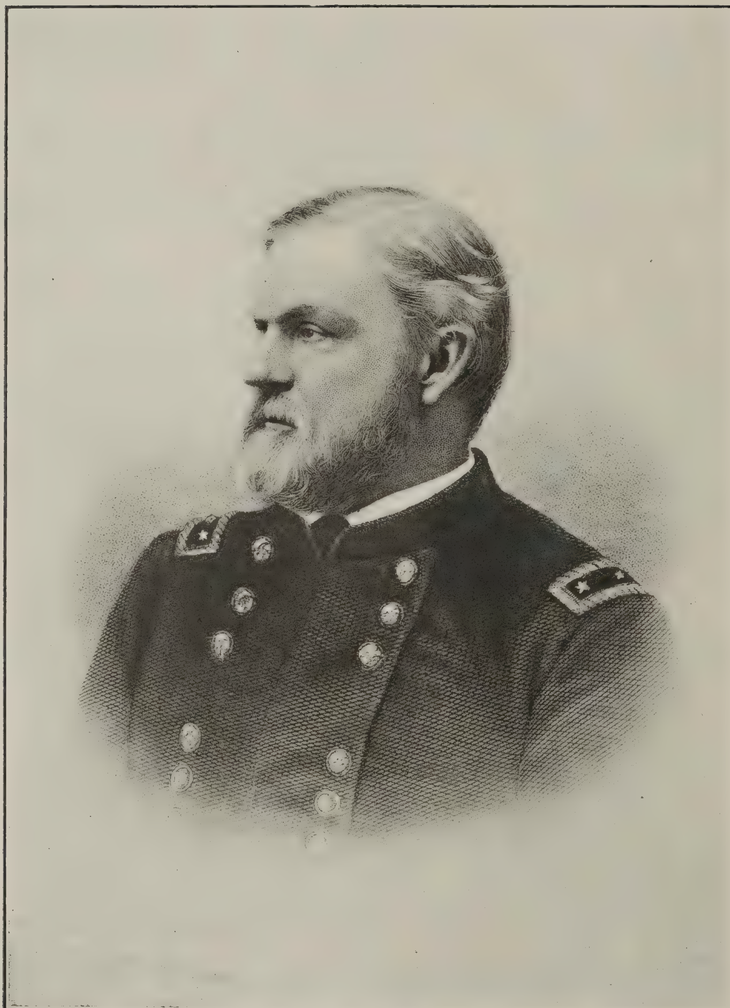


BRIG. GEN. JOHN NEWTON
CHIEF OF ENGINEERS, UNITED STATES ARMY
1884-1886
BORN 1823—DIED 1895

SEE PAGE 271

Dredges and Dredging in the Mobile District

BY

Mr. J. M. PRATT
Assistant Engineer

The subject of dredging, or excavating material, is an old one, the need for it being felt by the leading nations of each period of time since the world began. And yet until comparatively recent years a machine for doing this work had probably never been dreamed of. Nevertheless, the date when the simplest form of dredge was invented can not be given, simply because it can not be any more definitely located than that for any other primitive tool or apparatus.

The simplest form of dredge perhaps is the spoon apparatus, which consists of a strong iron ring or hoop, properly formed for making an impression on the soft matter at the bottom, so as to scoop it into a large leather bag attached to the ring and perforated with a number of small holes. The means for working it is a long handle, a suspended rope, and a crane or sweep-pole planted in a boat. This primitive apparatus was formerly used in the canals and ditches of the Netherlands.

The steam dredging machine now in common use is said to have been first applied by Boulton and Watt for use on the Weir at Sunderland, England, in 1796. It has a succession of strong iron buckets on an endless chain running on a frame, the lower end of which is vertically adjustable so as to regulate the depth at which it works. The material torn up and raised from the bottom by the buckets is discharged into barges or hoppers stationed close to the dredging vessel.

Dredge designing, unlike a great many other engineering problems, has not until very recently kept pace with the needs of mankind in that direction. This has been due, perhaps, to the inability to make the proper machinery, appliances and material. For instance steel castings and wire rope, two very important items on a modern dredge, were not manufactured until about the middle of the last half of the Nineteenth century. The need for more dredging, and for more efficient dredges, had gradually increased, result-

ing, in the latter part of the last century, in the invention of the hydraulic suction dredge. Improvements have been made in this style of dredge ever since, until now it has practically revolutionized dredging and made possible the construction of such ocean-going vessels, and consequently, the prosecution of such works as were never before dreamed of.

In the early days of civilization, before any form of dredge was invented, and later, while dredges were in a primitive state and the principle of modern dredging was slowly being unrolled by the onward march of stubborn necessity, those harbors with natural deep water were selected, and to those shipping flocked and around them the population centered. This continued until no suitable ones were left, and the ruling hand of commerce pointed with unerring aim to those with shallower depth, and ruthlessly demanded that they be fitted to receive their own. Then others and still others were selected, and hardly had they been prepared than greater depth and width were needed to accommodate the increased size of ships and ever-increasing trade, until the older ports themselves, which had at one time sufficient natural depth, were also called upon to improve their channels and harbors. This result was made necessary by each increase in the size of ships, the ships themselves being the result of the increase of commerce. This process has continued to the present time, with the limit not yet reached. Since the modern dredge with its great power and high efficiency is the result of commerce, and the recent rapid growth of commerce has been made possible by dredging, it is not surprising that so much importance is being attached to dredging and such rapid strides made in perfecting the design of the hydraulic suction dredge.

There are five principal types of dredges, namely, the dipper dredge, the ladder dredge, the grapple dredge, the sea-going hopper suction dredge, and the hydraulic suction dredge made without hoppers. The three last-named types are actually in use now in the Mobile district. There are at present (October 1, 1911), all told, six dredges in use in Mobile Harbor, two of which are clamshells, one a sea-going hopper suction dredge, and the other three hydraulic pipe line dredges. An interesting comparison of the efficiency of each type can therefore be obtained within the next ten or twelve months, especially as a portion or section of the channel in which each dredge must work contains material that is common to all.

There are three important channels in this district; viz,


DREDGED CHANNELS
OF THE
MOBILE DISTRICT
MOBILE
MOBILE BAY
Mobile Harbor Channel
BON SECOURS BAY
WEEKS BAY
MOBILE PT.
Mobile Bar Channel
SAND IS.
Dog R.
Fowl R.
CEDAR PT.
Grants Pass
DAUPHIN IS.
PETIT BOIS ISLAND
Horn Island Pass
Bar Channel
HORN IS.
ROUND IS.
PASCAGOULA
Pascagoula R.
Channel
MISSISSIPPI SOUND
Old Fort Bayou
BILOXI BAY
DEER IS.
Channel
BILOXI HARBOR
SHIP ISLAND
Ship Island Pass
CHANDELEUR IS.
GULF PORT
Gulf Port Channel
CAT IS.
Wolf River
ST. LOUIS BAY
Channel
Jordan River
GRAND IS.
Pearl Riv. Channel
East Pearl River
West Pearl River
LAKE BORGNE
Scale Of Statute Miles

Plate I.

Mobile Harbor ship channel, Gulfport ship channel, and the Pascagoula channel, the principal one of which is Mobile Harbor, with a projected depth of 27 feet. There are also smaller dredged channels at Biloxi, Wolf and Jordan rivers, and East Pearl River. It might be well to describe the characteristics of these channels sufficiently to enable a determination of the most efficient type of dredge to be used in each.

EAST PEARL RIVER, MISS.

Commencing at the one farthest west and taking them in order, we have first the East Pearl River channel at the extreme western end of the district. This channel is situated in Lake Borgne at the entrance to East Pearl River. It is about 1 1-5 miles long and connects the 9-foot contours, the shoal across which it extends having a depth of 7 feet. The project depth is 9 feet, and the width 200 feet. The material consists of mud, sand, and shells. Shoaling will probably take place in this channel at the rate of 30,000 cubic yards of soft mud per annum.

WOLF AND JORDAN RIVERS.

The channels at the mouths of these rivers are about 2½ miles apart and are situated in Bay St. Louis, an estuary of Mississippi Sound. The project provides for the formation of channels having a width of 100 feet and a depth of 7 feet, extending from the 7-foot curve in each river to the 6-foot contour in Bay St. Louis, as the mud beyond the latter point in either channel is so soft that there is a practical navigable depth of 7 feet. The material at the mouth of the Jordan River consists of soft mud and clay; and at the mouth of Wolf River, of soft mud. In redredging these channels only soft mud is encountered. These channels shoal at the rate of about 50,000 cubic yards per annum. The original depth at each locality was about 3 feet.

GULFPORT BASIN AND CHANNEL.

Proceeding eastwardly from Bay St. Louis along the Mississippi Coast the next channel is at Gulfport. The project for this improvement originally provided for the formation of an anchorage basin near the shore having a width of 1,320 feet and a length of 2,640 feet and a channel leading therefrom to the 19-foot contour in Mississippi Sound, both channel and basin to have a depth of 19

feet. The original depth over the area covered by the basin was from 2 to 7 feet. The original depth along the channel line increased from 7 feet at the inner end to 19 feet at the outer end, about 7 miles off. The material is mostly soft mud with some sand and clay. The project depth has been increased several times, the last modification being made by the River and Harbor Act of February 27, 1911, and authorized any depth that could be secured with available funds. This channel is in a very exposed locality. The seas roll high at times, making it hard on a pipe line, while the tidal current runs almost directly across the channel, causing it to fill at the rate of about 2,000,000 cubic yards per annum with a very soft mud having a specific gravity of 1.32. The average range of the tide along the Mississippi Coast is about 1¾ feet.

SHIP ISLAND PASS.

The entrance to Gulfport Harbor is made through Ship Island Pass, about 12 miles from Gulfport. Just outside of Ship Island there is a bar where originally there existed a depth of 21 feet. From funds appropriated in 1899, a channel was dredged through this bar 300 feet wide and 26 feet deep—the projected width and depth. The material consisted of sand and mud, the same kind the channel shoals with. It is about 2 miles long, connecting the 26-foot curves on either side. This is not a very ''rough'' bar to work on, and there is a good anchorage just inside of Ship Island. The rate of shoaling is not known, but it is believed to be about 75,000 cubic yards per annum, judging from the fact that it was dredged in 1900 and that in 1906 it was estimated that 365,000 cubic yards would have to be removed to restore it.

BILOXI HARBOR.

The channel at this locality is 1¼ miles long and extends from the 8-foot contour in Mississippi Sound to a small deep water pocket directly in front of the town. Originally, there was a depth of only 4 feet along the channel line. The channel was first formed in 1893, the projected depth being 8 feet and width 150 feet. It shoals at the rate of about 50,000 cubic yards per annum with mud, shells, and sand, mostly the latter.

PASCAGOULA CHANNEL.

This channel is 16½ miles long, about 10 miles of which is in the Pascagoula and Dog rivers, a tributary of the Pascagoula, and

the remainder in Mississippi Sound. The projected width, above the L. & N. Railroad bridge near the mouth of the river, is 150 feet and below this bridge it is 300 feet, while the projected depth throughout is 17 feet. A width of 225 feet, which is now considered ample, has been dredged below the bridge and the project completed above the bridge. The upper two-thirds of this channel is well protected but the rest, which extends almost across Mississippi Sound, is exposed where the sea becomes very "rough." The material in the river is mostly sand, but near its mouth and extending several miles into Mississippi Sound it is the toughest clay ever encountered in this district. It underlies a strata of mud. The seaward end of the channel consists mostly of soft mud. This channel shoals with soft mud at the rate of about 650,000 cubic yards per annum. Practically the entire shoaling takes place in Mississippi Sound, where the channel runs perpendicular to the general direction of the tidal current. This is the case with all the channels along the Mississippi Coast and is the real cause of their rapid shoaling. The depths on either side of the channel range from 3 to 17 feet, the former making it necessary in some portions to use a launch to shift the pipe line of hydraulic dredges.

HORN ISLAND PASS.

This Pass, which is about 12 miles from Pascagoula, is the entrance to Mississippi Sound from the Gulf of Mexico, and is only a few miles from the outer end of the Pascagoula Channel. It is to Pascagoula what Ship Island Pass is to Gulfport, and lies between Horn and Petit Bois islands. The original depth here was 18 feet and the material consisted of mud and sand. There is an inner and outer cut connected by deep water between the islands, the total length being 2 1-7 miles. The project provides for a width of 300 feet on the bar outside and 200 feet elsewhere in the Pass, with a depth of 21 feet throughout. The dredged channel shoals very slowly, the inner portion with mud and the outer or bar with sand. The project was completed in 1907.

MOBILE BAR, ALA.

This bar lies off the extreme entrance to Mobile Bay. The project provides for a channel 300 feet wide and 30 feet deep. This has been formed, with the exception of two shoals on the east side. The channel is about ¾ of a mile long, extending from the 30-foot contour on the inside to the 30-foot contour on the outside. The material is a hard, compact sand. The channel shoals with

the same material with a small proportion of mud. The amount of shoaling varies, but during the past eight years has averaged about 100,000 cubic yards per annum.

MOBILE HARBOR CHANNEL.

The Mobile Harbor dredged channel extends from Chickasaw Creek, 4.8 miles above the mouth of Mobile River, to deep water in the lower portion of Mobile Bay, a total distance of 33½ miles. The material along the upper 6 miles of this channel consists principally of sand and clay with some mud. Along the next 11 miles it is mud and sand with a strata of shells at the lower portion. The material along the remainder of the channel is composed of a soft blue mud having a specific gravity, as it lies on the bottom, of about 1.36. From the head of the channel in Mobile River to a point 10 miles below, the dredged material either has to be deposited on shore or towed several miles in scows, because the water on the edges of the channel is too shoal for loaded scows to get out. Mobile Bay varies in width from 8 miles at its upper end to 22 miles at a point just below its middle and to 3 miles at its lower end, measured from east to west. Accordingly, a dredge would be protected from storms, unless of unusual severity, while working in the channel in Mobile River, and be better protected while working in the upper portion of Mobile Bay than in the lower portion, thus making delays in dredging on account of weather conditions much less in the upper than in the lower bay and reducing them to a minimum in the river. All of these considerations make it difficult to form a comparison between two dredges working in this channel, unless engaged near the same locality at the same time. However, a fairly good comparison may be obtained of a clam shell and a hydraulic pipe line dredge by taking the records made in 1909 by the clam-shell dredge A and the hydraulic dredge B, both belonging to contractors and working under the same contract a few miles apart in middle and lower Mobile Bay. The yardage dredged represents the amount excavated from the theoretical section in either case, or the amount paid for and not the total amount removed. The time extends from April 1 to July 12, the hydraulic dredge working two days longer in July than the clam shell, which discontinued work on the 10th. The material in either case was mud, although that obtained by dredge A was much softer and more easily handled than where the hydraulic dredge was working during April and May. In June this dredge reached a point near where dredge A had been working, and her results were largely increased. These

dredges are both representative of their respective types and the material was deposited by each about 1,200 to 1,500 feet from the channel. Dredge A has an 8½-yard clam-shell bucket and Dredge B has a 20-inch centrifugal pump with a 22-inch suction and 20-inch diameter discharge. A table showing the amount dredged per month and the delays to each dredge follows:

Month.	Locality.	Cubic yards.	Time lost. Hrs.	Time lost. Min.
Dredge A.				
April 1 to 30	Between Beacons 4A and 2A	208,922	156	45
May 1 to 31	Between Beacons 4 and 2	195,231	179	30
June 1 to 30	Between Beacons 6A and 6	155,016	173	40
July 1 to 10	Between Beacons 8 and 6	70,567	55	55
Total		629,736	565	50
Dredge B.				
April 1 to 30	Between Beacons 10A and MBA	226,294	114	10
May 1 to 31	Between Beacons 10 and 8A	244,350	160	45
June 1 to 30	Between Beacons MBA and 8	410,821	66	45
July 1 to 12	Between Beacons 8 and 6A	142,576	44	30
Total		1,024,041	386	10

The time lost in either case does not include Sundays or legal holidays, but is a portion of the total effective working time. This work was done at a contract price of 9.95 cents per cubic yard, but during the last two contracts work has been done at this locality for a little less than 6 cents per cubic yard. Figuring on this basis and estimating the value of each outfit to be: $75,000 for dredge A and attendant plant; and $125,000 for dredge B and attendant plant, the value of each in earning power is as follows, omitting amounts less than $1:

Dredge A.

629,736 cubic yards dredged, at 6 cents		$37,784
Interest on $75,000 for three and one-third months, at 6 per cent	$1,250	
Depreciation at 10 per cent per annum, for three and one-third months	2,083	
Cost of operating dredge, three and one-third months	14,300	
		17,633
Amount earned		$20,151

Dredge B.

1,024,041 cubic yards dredged, at 6 cents		$61,442
Interest on $125,000 for three and one-third months, at 6 per cent	$2,083	
Depreciation at 10 per cent per annum, for three and one-third months	3,472	
Cost of operating dredge, three and one-third months	28,333	
		33,888
Amount earned		$27,554

Thus, in a little over three months, the hydraulic dredge earned about $7,500 more than the clam-shell dredge, though the latter was one of the best ever seen in this district. The hydraulic dredge experienced fewer delays in general, and could work as long during rough weather as the clam shell dredge, the latter, of course, having to quit work when it became too rough to land scows alongside. Previous to this work some doubt existed as to the wisdom of using a pipe line in lower Mobile Bay, but it is now believed that with suitable pontons—such as those in use by the Government dredges in this district, furnished with a sufficient number of anchors for emergency use and fitted with metallic connections on each section of pipe—the hydraulic dredge and attendant plant could work from one end to the other of the Mobile Harbor ship channel, especially if the best season of the year were selected for work in the most exposed locations. It is, in fact, the only type of dredge that can economically work at every place in the channel, inasmuch as the material dredged from the river has either to be placed ashore or towed several miles to a dumping ground, because the water on the banks on either side of the dredged channel along the upper 6 miles of Mobile Bay is so shoal (the depth over this portion ranging from 2 feet at the upper end to 10 feet at the lower end) that loaded scows have to go below this to obtain sufficient water to get out of the channel to the dumping ground. This would apply also to the sea-going, hopper, suction dredge. From the latter point, 6 miles below the mouth of the river, to a point about 3 miles above the 27-foot curve of depth in lower Mobile Bay, the natural depth gradually increases to 16 feet.

The total probable shoaling per annum in the inner channels in this district would amount to 3,780,000 cubic yards.

On the outer bars the shoaling per annum would be about 250,000 to 300,000 cubic yards.

The maintenance of these channels, together with some necessary additional work, would therefore require the services of two hydraulic pipe line dredges and an outer bar or sea-going dredge the greater part of the time. The hopper dredge could be used in the lower end of the inner channels when not needed on the bars.

The hydraulic pipe line dredge is mentioned because it is considered by far the most economical dredge that can work in all parts of all these channels. This style dredge has worked in practically all portions of all the inner channels in this district much more efficiently than the clam-shell dredge, even where conditions

were favorable to the latter. It is not believed that the hopper dredge would be as efficient for two reasons, first on account of the narrow cut it makes, causing a lack of efficiency through going back and forth over the same ground and doing a comparatively large amount of extra work to accomplish the desired result, and secondly because the actual time of pumping is greatly reduced by the time consumed in going to and from the dump.

GOVERNMENT DREDGES.

Two types of Government dredges have been used in this district, namely, the *Charleston*, a sea-going, hopper dredge borrowed from the Charleston district, and the *Pascagoula* and *Wahalak*, hydraulic pipe line dredges. It is not the intention here to give a detailed description of these dredges, as this can be obtained from specifications and drawings, but simply to mention the defects or the particularly good points that have appeared, together with such suggestions as to changes or additions as may seem best and of value in the construction of future dredges.

DREDGE CHARLESTON.

The *Charleston* has a 15-inch pump with drag on one side and a bin capacity of about 318 cubic yards. It is 122½ feet long, 30 feet wide, and is fitted with self-propelling machinery giving a speed, when light, of about 8 miles per hour. She draws 12 feet when light and about 17 feet when loaded, and carries one week's supply of coal and water. She was built in 1891, and was used in this district from August 15, 1906, to August 24, 1910, when she was recalled by the Charleston district.

Her work here consisted mainly in deepening and maintaining the channels through the outer bars connecting the Mobile Harbor, Pascagoula and Gulfport channels with deep water in the Gulf of Mexico. These localities are known as Mobile Outer Bar, Horn Island Pass, and Ship Island Pass, respectively. During portions of the time when engaged on the Mobile Outer Bar, it was too rough for her to work, and on such occasions while waiting for favorable weather she worked in the lower part of the Mobile Harbor ship channel. Here the material is very soft and light; so much so that it would not settle in the bins, and when once filled with a mixture of this material and water it was a question as to whether the extra material gained by pumping longer compensated

for the additional time and material lost overboard, as the overflow carried off a large percentage of material.

Another serious difficulty was encountered with this dredge which decreased considerably its output in this kind of material. It was found that when the drag was lowered below the surface of the material—that is, buried in the mud, that the pipe would continually choke and the drag would then have to be lifted in order to admit sufficient water to clear it. This, of course, would put a great deal of water in the bins which could not be disposed of and simply decreased the amount of material carried at each load. If the drag were kept near enough to the surface of the material to prevent choking a large percentage of water was admitted, and the

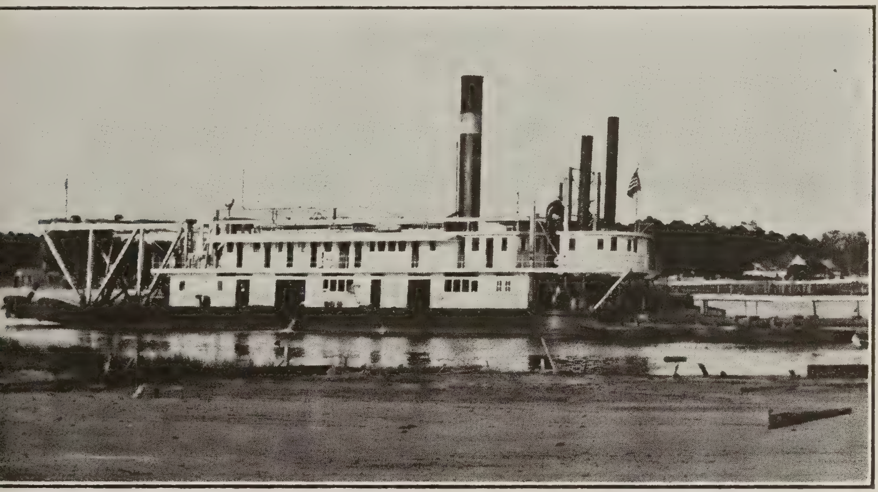


Fig. 1. Dredge *Pascagoula*. Side view, showing arrangement of house and machinery rooms underneath. (For description, see page 171.)

result was practically the same. The only way then to obtain a maximum amount of material in the shortest time was to bury the drag below the surface of the material, move slowly along the cut as before and admit just enough water in the drag to pump the material without choking it or having to raise the drag. This was accomplished by cutting a hole in the top of the drag and fastening thereon a pipe, $5\frac{3}{8}$ inches in diameter and 12 feet long, which extended up the outside of the main suction pipe and was fastened thereto. A valve was placed in the upper end of this $5\frac{3}{8}$-inch pipe, as this was always above the surface of the material on the

bottom, and just enough water admitted to enable the pump to work properly. When shifting the dredge from this locality to the outer bar this pipe was removed and a steel plate put in its place, filling up the hole in the drag.

The value of this pipe to the dredge when working in very soft

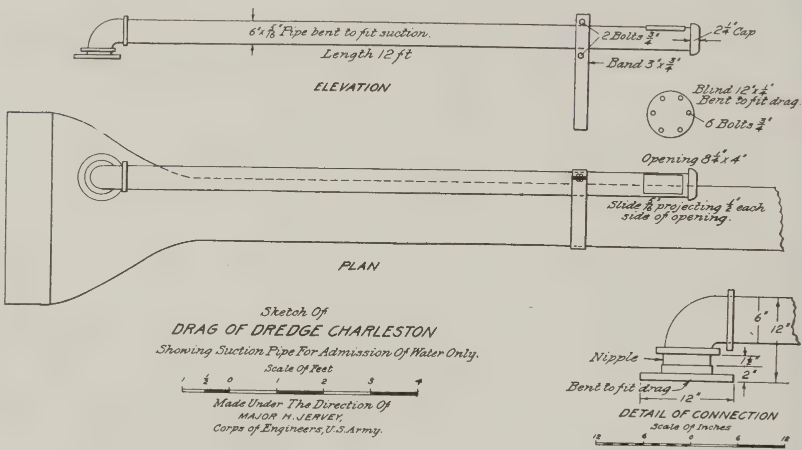


Fig. 2.

material may be seen from the following tables which show the work done just before and after its installation:

No. of loads.	Time loading.		Time dumping.		Going to dump.		Coming from dump.		Cubic yards
	Hrs.	Min.	Hrs.	Min.	Hrs.	Min.	Hrs.	Min.	
Previous to installation of auxiliary pipe.									
58	33	22	3	2	7	25	8	------	13,110
Average per load	----	34.52	----	4.86	----	7.67	----	8.28	226
Average time per 200 yards	----	30.55	----	4.30	----	6.79	----	7.33	200
	Total time per 200 yards dredged 48.97 minutes.								
After installation of auxiliary pipe.									
359	69	1	13	28	32	44	31	23	70,565
Average per load	----	11.56	----	2.25	----	5.47	----	5.25	196.6
Average time per 200 yards	----	11.76	----	2.29	----	5.57	----	5.34	200
	Total time per 200 yards dredged 24.96 minutes.								

A considerable portion of the loading time, as shown in the first part of the table, was consumed in clearing the suction and attempting to get a load of material by forcing out of the bins the water brought in when clearing the suction.

The reduction of the time going to and from the dump was due to a new cast-steel propeller having been installed instead of the old cast-iron one. This change was made at the same time the pipe was installed and made a considerable increase in the speed of the dredge. These two changes practically doubled the efficiency of the dredge in soft material.

The work done by this dredge and its cost are as follows:

Fiscal year.	Cubic yards.	Gross cost. [2]	Gross cost per cubic yard.	Material.
Horn Island Pass.				
			Cents.	
1907	353,230	$29,592.21	8.38	Mud and sand.
1908	184,017	16,233.90	8.60	Mud and sand.
1910	158,470	10,636.10	6.71	Mud and sand.
Total	695,717	$56,462.21	8.12	
Mobile Outer Bar and Channel.				
1908	185,683	$27,837.72	14.99	Very hard, compact sand.
1909	[4] 181,987	31,525.12	17.32	Very hard, compact sand and some soft mud.
1910	[3] 200,584	16,546.06	8.25	Very hard, compact sand and some soft mud.
1911 [5]	32,184	4,146.71	12.88	Very hard, compact sand.
Total	600,438	$80,055.61	13.33	
Ship Island Pass.				
1910	398,079	$12,537.94	3.15	Sand and mud, mostly sand.
Grand Total	1,694,234	$149,055.76	8.80	

[2]Includes cost of surveys, superintendence, and all repairs, as well as operating cost.

[3]Includes 78,944 cubic yards of mud removed from the Mobile ship channel.

[4]Includes 32,678 cubic yards of mud removed from the Mobile ship channel.

[5] Dredged from August 9 to 24, 1910.

Contract dredging work on Mobile outer bar has cost 30 cents per cubic yard. This work was done between 1903 and 1906.

In accordance with the Horn Island Pass project, which provided "that a contract or contracts can be made at a sum not to exceed the unit price of 11 cents per cubic yard ——," the dredging work was advertised in 1905, but no bids were received. Therefore, the contract price at this locality was greater than 11 cents per cubic yard, perhaps considerably.

On June 12, 1899, a contract for dredging across Ship Island Bar was awarded at 22½ cents per cubic yard. This work would probably be done at a considerably lower price now, but not as low as the work at Horn Island Pass, because a large portion of the latter work is on the inside of Horn Island and in soft mud. Of course, the *Charleston* seemed to do cheaper work at Ship Island, but that is due to the fact that she had remarkably good weather while there and had been put in first-class condition just before going there. Thus no large repair bills were charged against this work, in addition to which the dredge was much more efficient and had a far better crew than during a portion of the time at Horn Island. Balancing this work with contract dredging and assuming the contract cost at Horn Island Pass and Ship Island Pass to be 12 cents, the amount made by the United States plant is as follows:

Cost of Contract Work.

Mobile Outer Bar.		
488,816 cubic yards at 30 cents [4]		$146,644.80
Horn Island Pass.		
695,717 cubic yards at 12 cents		83,486.04
Ship Island Bar.		
398,079 cubic yards at 12 cents		47,769.48
Total		$277,900.32

Cost with United States Dredge "Charleston."

Gross cost [2]	$149,055.76	
Interest for 4 years at 6 per cent on $47,000 [3]	11,280.00	
Deterioration at 10 per cent per annum	18,800.00	
Total		$179,135.76
Amount made over contract work		$98,764.56

[2]This includes superintendence, surveys, office expenses, none of which should be included in a comparison with contract work.

[3]Value of plant.

[4]Does not include 111,622 cubic yards of mud removed from the Mobile Harbor ship channel. This should be included at a cost of about 5½ cents per cubic yard, the approximate contract cost at present.

DREDGE PASCAGOULA.

The dredge *Pascagoula* was built by the Ellicott Machine Company of Baltimore, Md., in 1909, under specifications prepared in this office under the direction and direct supervision of Maj. H. Jervey, Corps of Engineers, then in charge. She has a steel hull 150 feet long, 40 feet wide, 10½ feet deep and draws about 6¼ feet. Her ladder well is 30 feet long and 11½ feet wide. The cost of this dredge and attendant plant is shown below:

Cost of dredge	$143,600.00
Cost of outfit	1,806.13
Cost of attendant tug boat	21,500.00
Cost of attendant launch	3,215.40
Cost of pipe line, complete with all accessories	15,648.92
Cost of one coal and one water barge	3,999.00
Total	$189,769.45

LADDER.

The ladder of this dredge is 55 feet long and is of steel, the sides being solid plates. It rests on trunnions in the back part of the well and is raised by a cable attached to it at two points, one near the middle and the other near the outer end. The cutter is 6 feet 1 inch in diameter and 5 feet 7 inches long and is in two parts, one forward and one aft, bolted together. Each part is a solid steel casting. There are five knives. It is Ellicott Machine Company's special design, and has given excellent service in mud, sand, and hard, stiff clay. The cutter engine is placed directly on top of the rear end of this ladder and connected to the cutter shaft by a series of wheels and pinions.

This ladder is excellent, but it seems that three improvements could have been made by the builders at small cost, viz,

(a) The swinging sheaves attached to the outer end of the ladder can only turn through a comparatively small angle and when digging in a deep cut with steep sides the cable does not lead fair, thus cutting the sides of the sheaves. The brackets should have been higher so as to permit these sheaves to turn almost over.

(b) There should be two sets of rubbing plates and guides, one set as built on each side near the front, and the other set near the trunnions, in order that the vibration of the ladder, due principally to the cutter engine, which is horizontal, may be reduced to a minimum. This motion shakes the entire dredge, making it very dis-

agreeable and racking, especially when working in hard material. Blocks have been placed on each side near the after end of the ladder of the *Pascagoula* and *Wahalak* and have reduced this vibration considerably.

(c) The faces of timbers bolted to the ladder for the above purpose should be protected by strips of Swedish iron in order to prevent them from wearing. This has been done on the guides on the dredge *Pascagoula.* The first set had no protection pieces and had to be renewed in six months. The present set was then installed with soft iron strips partly countersunk in them and are now as good as ever after one and a half years' service.

LADDER LIFTING FRAME.

This frame on the *Pascagoula*, as may be seen from the photograph, is a heavy, cumbersome affair, concentrating all the load on either side of the well. It failed from a diagonal pull after a couple of months' service and some of the members, diagonal and athwartship, across the top had to be reinforced. The A-frame, as shown on the *Wahalak* is believed to be much better; it is light and distributes the load.

ENGINE ROOM AND MACHINE SHOP.

This is the most commodious and best arranged room on any dredge that has ever worked in this district. The main engine is a vertical, triple-expansion condensing engine, the cylinders being 12 inches, 20 inches and 33 inches in diameter with 20-inch stroke. It is run at a speed of 185 revolutions per minute. The main pump is a 20-inch centrifugal, single suction pump and the runner is 6 feet in diameter, having 5 blades.

It is believed that some very considerable improvements could have been made in the lower engine room where there is an ample number of pumps for every purpose.

(a) The condenser, which has 1,200 square feet of cooling surface, should have 1,400 or 1,500 square feet, as the vacuum in warm weather can be brought down to 17 inches only, while the circulating pump should be increased in size in order to reduce its speed, which is too great, thereby causing undue wear. It should have two suctions, one near the bottom of the hull and the other near the water surface.

(b) All pipes in the bilges and those connected with the salt water service, the fire system, and the blow-off pipes from the

boilers, should be of brass. Those pipes on this dredge were originally iron, but most of them have worn out and have been replaced with brass pipe. This, of course, has been done by the crew of the dredge at various times and no time has been lost on this account, but it would have been cheaper for the builder to have installed them in the first place.

(c) The foundation underneath the main engine is not solid enough to prevent considerable movement in the engine. This movement is being prevented by blocking up between the floors and the engine foundation and working holding-down bolts from the

Fig. 3. Dredge *Pascagoula*. Note complicated ladder frame and how weak it is laterally.

engine bed to the floor channels. There should also have been an additional column under the cylinders.

(d) The main bearing next to the pump is set diagonally instead of having each half come together in a horizontal position. This has given trouble, because the seam is partly underneath the shaft instead of being on the side where there is little pressure.

(e) The machine shop is fitted with a lathe, drill press, shaper, pipe machine, bolt machine, grindstone, twist drill grinder, and emery grinder, all of which are connected by shafting and belting

to the shop engine. In the forward part of this room and just under the lever room are the hauling and hoisting gear. All the friction and brake blocks are made by the crew of the dredge. The brake blocks, as supplied by the builder, consisted of four to the drum, each about 8 inches long. These did not hold properly and wore away rapidly and were replaced by two, each 30 inches long, cut to the circle of the drum. This was done by making a block long enough to reach from outside to outside of the pair of short ones, thus making one long solid one instead of the two short ones. They not only hold better, but last much longer.

(f) There is also an air compressor and necessary tools on this dredge, all of which have been used to great advantage, especially in reinforcing the ladder lifting frame and in replacing rivets in the ladder. It paid for itself on the former job alone.

(g) In the upper engine room, against the partition separating it from the boiler room are two electric plants, one a 15-kilowatt and the other a 7½-kilowatt machine. The dynamos are made by the General Electric Company. It would seem that one would be sufficient, but experience has taught that it is much better to have the two. For instance, a short while after the dredge commenced operating, the larger of these machines broke down and several weeks elapsed before it was operating again, as the factory had to supply the broken parts. The boat would have been without anything but the unsatisfactory oil lights during this time had there been only one machine, and this added to the risk of burning oil is something to be considered. Another argument in favor of two dynamos is the comfort of the men in this climate. Were there only one machine it would be shut down in the day time, and men working in dark places in the hold would be at a disadvantage. In addition to the above, the small machine operates the fans during the day in hot weather—an important item in this climate, especially for those men who work all night and have to sleep in the day time. It is believed that such a seeming luxury is repaid over and over again by a willing, satisfied crew.

Two 14-inch searchlights, each of 20-ampere capacity, on a 2½-foot stand, were installed on top of the lever room. These were found to be very weak and were exchanged after about one and one-half years for one 18-inch 35-ampere light on a 52-inch stand. This is sufficient for all purposes.

BOILER ROOM.

Steam is supplied by two Scotch marine boilers, each containing

1,600 square feet of heating surface and 50 square feet of grate surface. They are each fitted with 3-inch, No. 11 gauge, charcoal iron tubes and are built for a working pressure of 180 pounds. They were built by the Maryland Steel Works and are very economical and furnish an ample supply of steam.

Some improvements could have been made in this room and one item could have been left out, as follows:

(a) The safety valve is up over the boilers and condensed steam drips from it, injuring the covering on the boiler. This should be placed to one side.

(b) The donkey boiler could be left out altogether, as it has never been used. There is none on the *Wahalak*.

(c) All steam joints should be fitted with copper gaskets.

(d) The main stop valves should be arranged so they could be shut off from the roof.

VALVE IN SUCTION PIPE.

This is a piece of metal in length more than twice the diameter of the suction pipe and is forward of the stone box. There is a hole 20 inches in diameter in it near the lower end. A groove on either side and at the bottom is provided for it to slide in. When the dredge is pumping, of course, this valve is raised. When it is desired to shut the flow off it is necessary to lower it. The groove in the bottom in the meantime has become so filled with material that it is difficult, and sometimes impossible, to close it, especially when working in hard material. Even the jet intended for this purpose will not clear it. The hole in this valve, if made in this way, should be placed near the top, so that when pumping the valve would be lowered and the groove closed. In the case of this dredge, this groove was closed at bottom with cement and the bottom half of the valve (that containing the hole) cut off and no trouble has occurred since.

ICE MACHINE.

An ice machine built by the Brunswick Refrigerating Company, 1 ton size, has been installed in the after part of the hold of the dredge. This is the worst place it could have been installed in, as it is in a dark hold and too far from the cold storage. It should have been placed on the main deck, as close to the cold storage as possible. A similar ice machine on the dredge *Wahalak* placed on the main deck aft gives much better results; it not only keeps the

cold storage in proper condition, but makes enough ice for the dredge and attendant tug.

SPUDS.

The spuds are 60 feet long and 32 inches in diameter and are placed 12 feet 9 inches apart, one being in the center and 16½ feet from the stern. The discharge pipe, which is 20 inches in diameter, is carried through the stern of the dredge on the port side about 16 feet from the center line. It is very advantageous to have the center spud so near the stern, as it gives such a small radius through which the discharge pipe and nearby pontons have to move when the dredge is swinging, thus making the swinging much easier, especially in a strong current.

It is an excellent idea to protect the spuds from wear. The first ones were used in their natural state and lasted only about six months. The next were protected by Swedish iron strips, eight for the center spud and six for the wing spud. They were 1 by 3 inches and were countersunk in the spud, being placed vertically along the entire length of the portion subject to wear. These spuds have not worn a particle.

GUARDS.

The fender pieces are placed about 5½ feet above the bottom of the boat. They should be placed 1 foot higher, as they are under water most of the time—a serious matter when working in teredo infested waters. They have been copper painted and covered with galvanized iron, but will have to be looked after quite often.

STONE BOX.

A water-tight compartment should be built around the stone box, from the hull to about 3 feet above the top of the floors, in order to confine the material when it is opened. This was done on the dredge *Pascagoula* by using material 3 inches thick and caulking same.

CASTINGS.

All sheaves, pinions, gear wheels, and drums connected with the hauling, hoisting and cutter machinery, should be of cast steel brass bushed.

SUCTION MOUTH PIECE.

It is believed that this should be much heavier and of cast steel.

HOUSE.

In construction and arrangement the house of the dredge *Pascagoula* excels that on any dredge ever seen in this district. It is $1\frac{1}{2}$ feet lower than the one the *Wahalak* and this, together with the shallower hull of the former, makes a surprising difference in the handling and swinging of the two dredges, as the increased wind surface of the house and hull of the *Wahalak* gives a considerable advantage in favor of the *Pascagoula* when there is a

Fig. 4. Dredge *Pascagoula*. Note location of spuds with officers' quarters back of them.

stiff breeze. The depth of the hull and the height of the house on the dredges *Pascagoula* and *Wahalak* are as follows:

	Depth of hull.	Distance from main to saloon deck.	Distance from saloon deck to top of cabin.
	Feet.	*Feet.*	*Feet.*
Pascagoula	$10\frac{1}{2}$	9	8
Wahalak	$11\frac{1}{2}$	10	$8\frac{1}{2}$

Two principal defects, however, developed and were remedied soon after the dredge *Pascagoula* commenced work. One was the weakness of the stringers running the entire length of the deck house. They showed signs of weakness in the long spans in the engine and boiler rooms. They were immediately reinforced with trusses extending from column to column. The other defect was leaking around the skylights. They were poorly constructed, and it seemed this could not be remedied without rebuilding. To avoid this and also to keep the glare, which was very annoying, out of the rooms below, a covering of pipe stanchions and thin matched plank was built high enough over each one to allow them to open. Canvas was also placed on top of the plank. These look well and have given perfect satisfaction. The smoke stack from the galley stove should lead into the main stack on the roof.

The general arrangement or plan of the saloon deck is as follows:

Beginning at the forward end, first there is the lever room, the floor of which is raised 4 feet 3 inches above the saloon deck. This room is 18 feet by 10 feet 3 inches, and is surrounded by glass windows and doors. Between the lever room and the main cabin is a passageway 3 feet wide. The cabin for the officers is 32 feet 4 inches long and 28 feet wide. There are four staterooms on either side, all being provided with double bunks except the forward ones, which are intended for the master and chief engineer. Back of these rooms there is a bathroom and closet on one side and a wash room, containing three lavatories and a closet, on the other. Between these two rows of rooms and in the center of the space there is—beginning at the forward end—first, the office, which is 11 feet by 9 feet, and just behind it the officers' dining room, which is about 23 feet long by 11 feet wide.

Back of the officers' quarters is a passageway extending across the boat and 3 feet wide. The space—28 feet 6 inches long and 28 feet wide—between this passage and the 3-foot 4-inch passage next aft, is taken up as follows:

Port side; crews' dining room, 8 feet 6 inches by 20 feet 6 inches; and the crews' wash room and toilet, 10 feet by 7 feet 6 inches.

Starboard side: cook and waiters' room, storeroom, linen room and waiters' room.

In center of space between these rows of rooms is the kitchen, in which is the cold storage. It opens directly into the crews' dining room on one side, the storeroom on the other and the hall

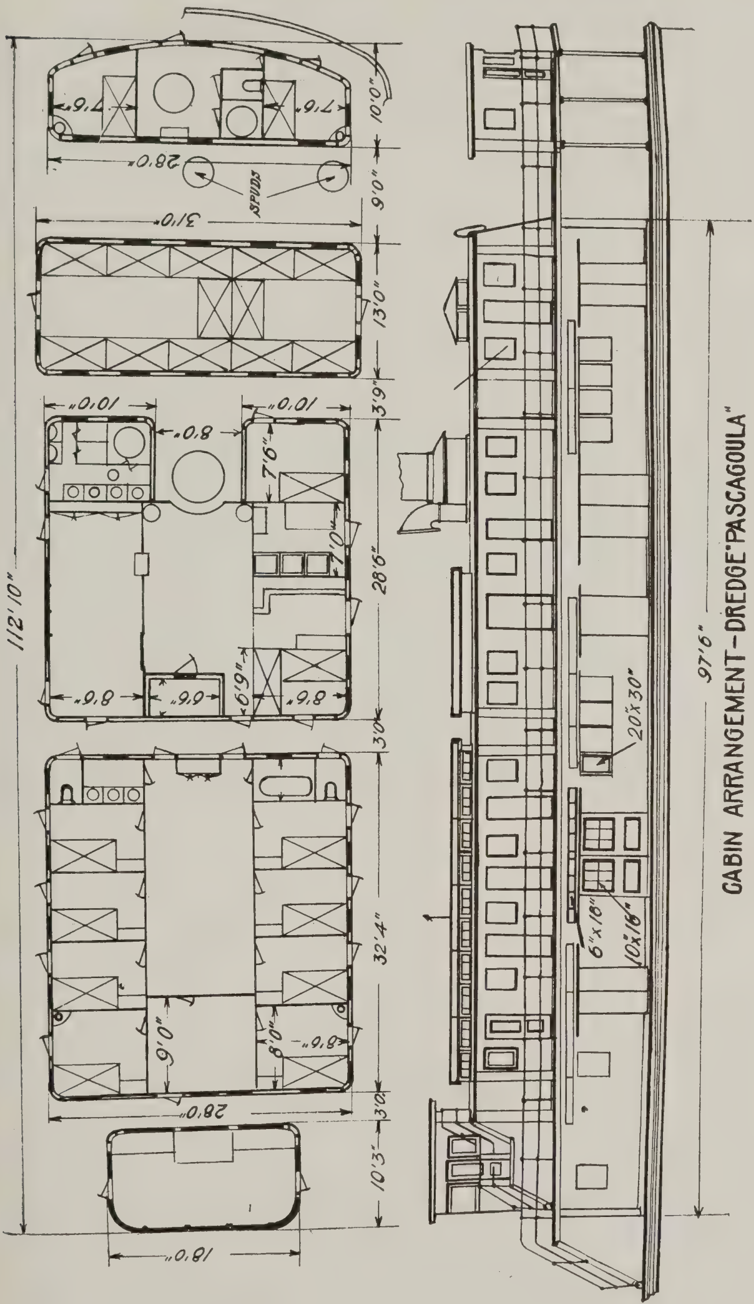

CABIN ARRANGEMENT-DREDGE"PASCAGOULA"
112'10"
97'6"
SPUDS
28'0"
31'0"
18'0"
10'0"
9'0"
13'0"
3'9"
28'6"
3'0"
32'4"
10'3"
7'6"
7'0"
6'9"
8'6"
20"x30"
6"x18"
10"x18"

Fig. 5

or passageway forward. The kitchen is 20 feet 6 inches long and 11 feet wide. The space just back of the kitchen is taken up by the smokestack.

Back of the next passageway is the crews' room. It is a little wider than the rest of the house, being 31 feet wide by 13 feet in length. Back of this there is a passageway where the main house ends. The spuds take up the next space, aft of which are quarters for visiting officers. The officers' quarters are separate from the main house, and are connected therewith by two walkways—one on either side—each 4 feet wide, and are supported by pipe stanchions and braces, so as to leave the main deck below on the after part of the dredge as clear as possible for handling lines, etc. The visiting officers' quarters consist of two bed rooms, one on each side, and a dining or sitting room, closet and shower bath. These quarters have been criticised as being supported in a frail manner and located in the wrong place, etc., but no weakness has developed from the manner of support and it is not in the way of any necessary work on the main deck. It is not believed that any better location could have been found.

In general, the finish and arrangement of this house is excellent. A movable wooden grating 20 inches wide has been placed over all walkways in order to protect the canvas covering.

PIPE LINE.

The floating pipe line is supported by strongly built wooden pontons, each 25 feet 4 inches long, 4 feet 6 inches wide and 5 feet deep, with rakes on each end for easy handling. They are narrow and high, in order, first, that the least resistance may be offered to the current or sea, and secondly to place the pipe line high enough so that seas will pass under it instead of breaking against it broadside. Large timber winches with wire cables and anchors attached are placed on the pontons here and there, as the occasion requires, in order to keep the pipe line straight and hold it during a blow. The sections of pipe are 40 feet long and consist of riveted boiler steel ⅜-inch thick. Each section of pipe is supported by two pontons. The connections are made with rubber sleeves. They are held on the pontons by wooden saddles, which are clamped together and fastened to the pontons with bolts. The pontons draw about 23 inches when light, and about 32 inches when the pipe is full. This pipe line recently rode out a severe storm at Gulfport, Miss., when the tide rose 6 feet and the sea was so rough that several

vessels and considerable property was damaged. The pipe line is about 1,200 feet long. The pontons are protected from the teredo below water by creosoted sheathing. The cost of this pipe line complete, with 300 feet of shore pipe and a full set of rubber dredging sleeves, was $15,648.92.

The attendant plant connected with this dredge consists of a tug boat, a launch—used in shifting anchors where the water is too shoal for the tug boat—and a coal and water barge.

Fig. 6. Pipe line, showing one 40-foot length supported on two pontons, one of which is fitted with a windlass.

The following data regarding the work accomplished by this dredge at the localities named below may be of interest:

Detailed Statement.

Improvement.	Material removed	Operating cost. Total.	Operating cost. Per cubic yard.	Total cost.	Gross cost per cubic yard.
	Cubic yards.		*Cents.*		*Cents.*
Pascagoula River, Mississippi	1,233,653	$56,519.88	4.58	$66,002.09	5.35

NOTE: Measurements in place.

DISTRIBUTION OF EFFECTIVE WORKING TIME.

	h.	m.
Pumping	3,620	48
Handling pipe line	178	45
Handling swinging wires	136	45
To and from wharf	37	00
Placing dredge	8	57
Waiting for vessels to pass	1	40
Total	3,983	55

DISTRIBUTION OF TIME LOST WHILE IN COMMISSION.

	h.	m.
Changing location of plant	143	50
Bad weather	135	04
Washing boilers and ordinary repairs	1,019	18
Extraordinary repairs	328	30
Other causes	101	23
Sundays and holidays	1,080	00
Total	2,808	05
Total time to be accounted for (September 21 to June 30)	6,792	00

MONEY EXPENDED.

Pay roll	$23,553.38
Fuel	11,320.17
Water	255.88
Supplies, subsistence	5,551.60
Supplies, engine room	3,702.46
Other supplies	5,319.22
Renewals or additions to outfit	4,775.64
Ordinary repairs	1,320.70
Laundry, ice, miscellaneous expenses	720.83
Total operating cost	$56,519.88
Extraordinary repairs	1,197.24
Office expenses, superintendence, surveys, etc.	8,284.97
Total cost of dredging	$66,002.09

The above information was taken from the Annual Report of the Chief of Engineers for 1910.

Detailed Statement.

Improvement.	Material removed.	Operating cost. Total.	Operating cost. Per cubic yard.	Total cost.	Gross cost per cubic yard.
	Cubic yards.		*Cents.*		*Cents.*
Pascagoula River, Miss.	449,713	$33,316.13	7.41	$40,724.99	9.06
Biloxi Harbor, Miss.	109,088	3,830.20	3.51	4,222.49	3.87
Wolf and Jordan rivers, Miss.	111,477	5,221.24	4.68	5,804.81	5.21
East Pearl River, Miss.	128,843	4,432.63	3.44	7,059.73	5.48
Gulfport Harbor, Miss.	919,892	34,644.60	3.77	39,596.91	4.30
Total	*1,719,013	$81,444.80	4.74	$97,408.93	5.67

Measurements in place.

*This is the total yardage dredged from within the *limits* of the project, from 1 to 2 feet being allowed for over depth.

The total yardage dredged, inside and outside the project, was 2,013,555.

DISTRIBUTION OF EFFECTIVE WORKING TIME.

	h.	*m.*
Pumping	3,919	57
Handling pipe line	493	06
Handling swinging wires	235	30
Placing dredge	102	00
Waiting for vessels to pass	27	30
Total	4,778	03

DISTRIBUTION OF TIME LOST WHILE IN COMMISSION.

	h.	*m.*
Changing location of plant	356	30
Bad weather	170	50
Washing boilers and ordinary repairs	1,796	04
Other causes	242	33
Sundays and holidays	1,416	00
Total	3,981	57
Total time to be accounted for	8,760	00

MONEY EXPENDED.

Pay roll	$35,411.28
Fuel	14,789.73
Water	173.08
Supplies, subsistence	9,243.70
Supplies, engine room	3,786.36
Total	$63,404.15

Brought forward	$63,404.15
Other supplies	8,172.34
Renewals or additions to outfit	5,609.87
Ordinary repairs	3,683.31
Laundry, ice, miscellaneous expenses	575.13
Total	$81,444.80
Extraordinary repairs	3,191.55
Office expenses, superintendence, surveys, etc.	12,772.58
Total	$97,408.93

The above information was taken from the Annual Report of the Chief of Engineers for 1911.

During July, August, and September of the present fiscal year, this dredge accomplished the following in Gulfport ship channel where the material is very soft:

Month.	Place measurement.	Average width of cut.	Depth before dredging. Average.	Depth after dredging, Average.
	Cubic yards.	*Feet.*	*Feet.*	*Feet.*
July 6 to 31	578,783	200 to 225	15 to 16	$22\frac{1}{2}$
August 1 to 31	755,624	200	16 to 19	22 to 24
September 1 to 15	226,578	200 to 225	17 to 21	21 to 23

DREDGE WAHALAK.

This dredge was also built by the Ellicott Machine Company of Baltimore, Md., and was purchased by this office and commenced working in Mobile Harbor, about 3 miles below the mouth of Mobile River, in March, 1911. She has about the same attendant plant as the dredge *Pascagoula*, and the same kind of pipe line. The following data regarding this dredge and her work to June 30, 1911, was taken from the Annual Report of the Chief of Engineers:

Detailed Statement.

Improvement.	Material removed.	Operating cost.		Total cost.	Gross cost per cubic yard.
		Total.	Per cubic yard.		
	Cubic yards.		*Cents.*		*Cents.*
Harbor at Mobile, Ala.	*1,005,935	$23,434.81	2.33	$24,700.65	2.46

Measurements in place.

*This material was obtained from within the limits of the project, allowing 1 foot for over-depth. The total number of cubic yards removed, inside and outside the project limits, was actually 1,128,777.

DISTRIBUTION OF EFFECTIVE WORKING TIME.

	h.	m.
Pumping	1,316	01
Handling pipe line	121	20
Handling swinging wires	47	36
Placing dredge	6	55
Waiting for vessels to pass	1	33
Total	1,493	25

Fig. 7. Dredge *Wahalak*. Front view. Note cutter and solid side of ladder frame. Note, also, A-frame for raising cutter and frame just back of it supporting the line from stern of boat to the A-frame.

DISTRIBUTION OF TIME LOST WHILE IN COMMISSION.

	h.	m.
Bad weather	13	15
Washing boilers and ordinary repairs	183	53
Extraordinary repairs	204	44
Other causes	114	43
Sundays and holidays	360	00
Total	876	35
Total time to be accounted for (March 24 to June 30)	2,370	00

MONEY EXPENDED.

Pay roll	$7,806.76
Fuel	5,421.02
Supplies, subsistence	2,479.18
Supplies, engine room	1,845.42
Other supplies, rope, sleeves	4,913.56
Launch supplies	142.02
Renewals or additions to outfit	551.47
Ordinary repairs	122.54
Laundry, ice, miscellaneous	152.84
Total operating cost	$23,434.81
Extraordinary repairs	26.40
Office expenses, superintendence, surveys, etc.	1,239.44
Total cost of dredging	$24,700.65

From July 1 to September 30, 1911, she accomplished the following:

Month.	Average width of cut.	Average depth before dredging.	Average depth after dredging.	Place measurement.
	Feet.	*Feet.*	*Feet.*	*Cubic yards.*
July	140	5	29 to 31	392,037
August	140	4½	28 to 32	296,027
September	135	3 to 10	14 to 29	93,870
Total				781,934

The small amount dredged in September is due to the fact that the dredge was under semi-annual repairs from September 1 to 7, and also because several large wrecks and numerous logs were encountered, making it necessary to stop operations every few minutes over a considerable part of the time.

The amount dredged, as shown above, was removed from within the limits of the project, allowing 1 foot for over-depth. The total amount dredged, inside and outside project limits, was 861,945 cubic yards.

The principal differences between this dredge and the *Pascagoula* are as shown on opposite page.

	Pascagoula.	*Wahalak.*
Depth of hull	10½ feet	11½ feet
Length of ladder (to center of cutter)	55 feet	70 feet
Ladder lifting frame	Built up and supported on each side of well (as per photograph).	A-frame.
Point of attachment of cable for raising ladder	At the outer end and in the middle	At the outer end.
Main engine, vertical triple expansion, 20-inch stroke	Cylinders 12-inch, 20-inch, 33-inch	Cylinders 14-inch, 22½-inch, 40-inch
Revolutions per minute of main engine	180	145
Main pump, 20-inch centrifugal, single suction with inclosed runner and discharge at bottom	72 inches in diameter 5-blade runner	96 inches in diameter, 6-blade runner
Main boilers	2	4
Donkey boilers	1	0
Electric machines	One 15 K. W. and one 7½ K. W.	One, 15 K. W.
Ice machines, 1-ton size	In hold aft	On main deck aft.
From main to saloon deck	9 feet	10 feet.
From saloon deck to top of cabin	8 feet	8½ feet.
Method of raising spuds	Cable from drum direct to carriage on roof, then back to gallows frame	Cable from drum through hull to stern, then up to gallows frame.
Height of gallows frame for raising spuds	32½ feet	36 feet.

Generally, the same improvements could have been made on this dredge as mentioned in connection with the *Pascagoula,* except that the condenser is of sufficient size, the main engine foundation is more solid, the ice machine is better located, and certain smaller

details improved. The additional improvements that could have been made are as follows:

(a) The discharge pipe in the stern of the boat should project at least 1 foot farther out, as it is so short that some difficulty is experienced in putting rubber dredging sleeves on it.

(b) A boom of suitable strength should be placed on the stern of the dredge to support the nipples and stern connections between the dredge and the first ponton.

(c) The solid rings placed on the spuds for raising and lowering them have been replaced by clamps, consisting of two half-bands with lugs on each end and bolted together, fitting tight around the spuds. The solid iron rings which pressed against the spud at only two points in raising, cut the fibre of the wood, causing considerable wear, and besides they did not always hold.

(d) The condenser should be provided with copper discharge pipe and copper pipe between circulating pump and condenser. This would cost about $250, while the cost of the iron pipe would probably be only one-third as much but would have to be renewed, perhaps, every two years if in salt water, and a delay amounting to many times the value of the copper pipe might sometimes be caused from the dredge being put out of commission while replacing this pipe.

(e) There should be two suctions to the main injector, one near the bottom of the hull and the other about 1 or 2 feet below the surface of the water, as the dredge *Pascagoula* has had considerable trouble, especially when working in two channels where the projected depth was only 7 feet.

(f) The cabin on this dredge has been very poorly arranged. The space has been badly divided and does not accommodate the crew that it should, nor is it convenient. The kitchen is excessively hot, the cook being hardly able in warm weather to stay in it long enough to prepare meals. It should connect directly with either side of the house, as on the dredge *Pascagoula*, to admit draft. The arrangement on the *Pascagoula* is an excellent one, there being but one suggestion for improvement, which applies also to the *Wahalak*, viz, the crews' wash room and closet should be placed in the after compartment of the hull, which could be nicely arranged for the purpose, the head room being sufficient for natural drainage. This would not be inconvenient for them, as they could wash before coming upstairs, and would give more room on the upper deck, besides dispensing with a nuisance caused

by leaks from this room which injure the boilers and make conditions generally more or less unsanitary.

(g) Awnings of No. 6 canvas with iron pipe supports have been placed over all walk ways on the upper deck of both the *Wahalak* and *Pascagoula*. These extend out to the line of the rail, on which they are supported by pipe stanchions.

(h) The screens on both these dredges are of very light copper

Fig. 8. Dredge *Wahalak*. Stern view. Note spuds with vertical iron strips, and frame for raising them. Note, also, where pipe line comes out of side of dredge.

wire and, consequently, do not last very long. They should be of about 23-gauge copper wire, fourteen wires to the inch.

DREDGE BARNARD.

The United States dredge *Barnard* was in this district from June 28, 1907, to December 16, 1910. This is a self-propelled pipe line dredge with center drag 7 feet wide. Her discharge pipe is 36 inches in diameter. It is floated on steel pontons and kept on one side of the channel by means of a baffle plate, against which the material strikes as it comes out of the end of the pipe while the dredge is moving up and down the channel. Owing to the narrow,

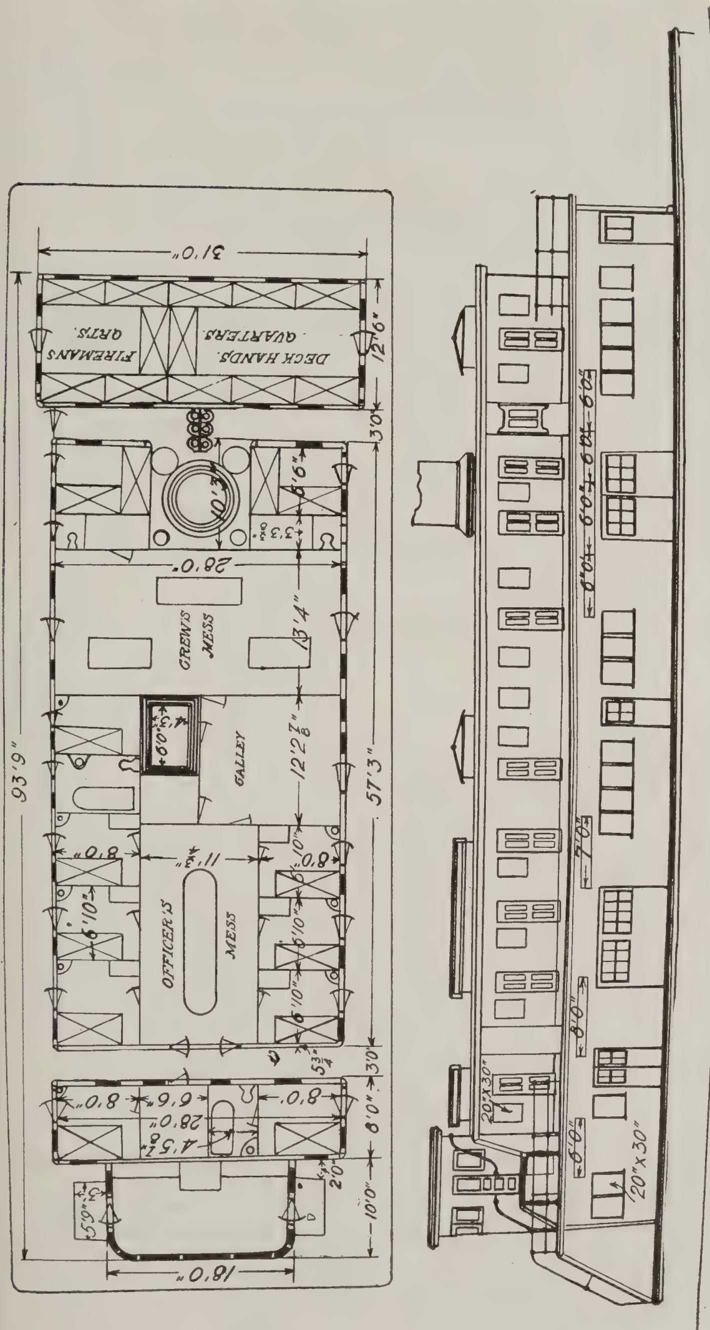
crooked cut this dredge makes, it can not be used to advantage in any material except the very softest kind—so soft, in fact, that it will flow readily and fill up the cut made. It is not a success even in this kind of material, as there is necessarily a great loss of efficiency in operating that style of dredge. The only place in the district where the *Barnard* was used with any degree of success was in the channel at Gulfport, Miss. Her record in this district is as follows:

Month.	Location.	Yardage.	Field cost.	Cost per cubic yard.
1907				
June		Dredge reached District June 28th.	$848.80	
July	Pascagoula River, Miss.	1,837 (Worked 3 days.)	5,340.56	
August	Pascagoula River, Miss.	64,643	5,248.84	$0.081
September	Pascagoula River, Miss.	47,593	5,014.45	.1054
October	Gulfport, Miss.	167,869	6,518.43	.0388
November	Gulfport, Miss.	199,252	6,923.87	.0347
December	Gulfport, Miss.	253,651	6,277.93	.0247
1908				
January	Gulfport, Miss.	251,563	7,098.03	.0282
February	Mobile Harbor, Ala.	29,209 (Worked 10 days.)	6,603.64	.2261
March	Fort Morgan, Ala.	Filling reservation	7,093.46	
April	Fort Morgan, Ala.	Filling reservation	7,093.80	.1486
Do	Gulfport, Miss.	47,750 (Worked 10 days.)	6,965.19	.0825
May	Gulfport, Miss.	84,427 (Quarantined portion of month.)		
June	Gulfport, Miss.	261,548	6,755.70	.0258
Total for fiscal year 1908		1,409,342	$77,782.70	.0552

DETAILED STATEMENT. (*Fiscal Year 1909.*)

Improvement.	Material removed.	Operating cost.		Total cost.	Gross cost per cubic yard.
		Total.	Per cubic yard.		
	Cubic yards.		*Cents.*		*Cents.*
Pascagoula River	586,353	$54,524.92	9.30	$67,204.13	11.46
Gulfport Harbor	1,589,620	45,403.09	2.86	59,128.79	3.72

NOTE: All measurements in place.



CABIN ARRANGEMENT–U.S. DREDGE "WAHALAK".
Fig. 9.

DISTRIBUTION OF EFFECTIVE WORKING TIME.

	h.	m.
Pumping	2,760	15
Going to and from wharf and anchorage	77	00
Placing dredge	212	15
Total	3,049	30

DISTRIBUTION OF TIME LOST WHILE IN COMMISSION.

	h.	m.
Changing location of plant	175	45
Bad weather	338	00
Washing boilers and ordinary repairs	1,922	30
Extraordinary repairs	1,028	45
Other causes	712	00
Sundays and holidays	1,533	30
Total	5,710	30
Total time to be accounted for	8,760	00

MONEY EXPENDED.

Pay rolls	$40,439.94
Fuel for boilers	24,605.18
Fuel for galley	129.09
Water	53.75
Supplies, subsistence	11,984.13
Supplies, engine room	4,816.57
Other supplies	3,047.27
Renewals or additions to outfit	8,484.57
Ordinary repairs	5,441.20
Laundry, ice, miscellaneous expenses, etc.	926.31
Total operating cost	$99,928.01
Extraordinary repairs	15,499.37
Office expenses, superintendence, surveys, etc.	10,905.54
Total cost of dredging	$126,332.92

DETAILED STATEMENT. (*Fiscal Year 1910.*)

Improvement.	Material removed.	Operating cost.		Total cost.	Gross cost per cubic yard.
		Total.	Per cubic yard.		
	Cubic yards.		*Cents.*		*Cents.*
Gulfport Harbor, Mississippi	3,005,270	$91,192.24	3.03	$110,168.63	3.67

NOTE: All measurements in place.

DISTRIBUTION OF EFFECTIVE WORKING TIME.

	h.	m.
Pumping	2,278	27
Going to and from wharf or anchorage	55	45
Placing dredge	216	48
Waiting for vessels to pass	10	41
Total	2,561	41

DISTRIBUTION OF TIME LOST WHILE IN COMMISSION.

	h.	m.
Changing location of plant	19	30
Bad weather	231	35
Washing boilers and ordinary repairs	725	57
Extraordinary repairs	321	00
Other causes	272	17
Sundays and holidays	1,004	00
Total	2,574	19
Time lost on account of not working at night	3,624	00
Total time to be accounted for	8,760	00

Note: Double crew maintained until September 2.

MONEY EXPENDED.

Pay rolls	$33,931.37
Fuel for boilers	25,464.23
Fuel for galley	132.00
Supplies, subsistence	10,133.49
Supplies, engine room	5,442.70
Other supplies, paints rope, etc.	2,922.39
Renewals or additions to outfit	8,455.50
Ordinary repairs	3,845.21
Laundry, ice, miscellaneous expenses, etc.	865.35
Total operating cost	$91,192.24
Extraordinary repairs	5,760.11
Office expenses, superintendence, surveys, etc.	13,216.28
Total cost of dredging	$110,168.63

DETAILED STATEMENT. (*Fiscal Year 1911.*)

Improvement.	Material removed.	Operating cost.		Total cost.	Gross cost per cubic yard.
		Total	Per cubic yard.		
	Cubic yards.		*Cents.*		*Cents.*
Gulfport Harbor, Mississippi	1,156,938	$47,968.79	4.15	$54,927.19	4.75

NOTE: All measurements in place.

DISTRIBUTION OF EFFECTIVE WORKING TIME.

	h.	*m.*
Pumping	1,122	28
Going to and from wharf or anchorage	14	10
Placing dredge	90	43
Waiting for vessels to pass	5	43
Total	1,233	04

DISTRIBUTION OF TIME LOST WHILE IN COMMISSION.

	h.	*m.*
Changing location of plant	----	--
Bad weather	4	07
Washing boilers and ordinary repairs	313	40
Extraordinary repairs	6	00
Other causes	69	09
Sundays and holidays	402	00
Total	794	56
Time lost on account of not working at night	2,028	00
Total time to be accounted for (December 16, inclusive)	4,056	00

MONEY EXPENDED.

Pay rolls	$17,215.40
Fuel for boilers	14,916.26
Fuel for galley	84.00
Supplies, subsistence	5,127.55
Supplies, engine room	1,915.70
Other supplies, paints, ropes, etc.	2,961.97
Renewals or additions to outfit	3,888.08
Ordinary repairs	1,470.25
Laundry, ice, miscellaneous expenses, etc.	389.58
Total operating cost	$47,968.79
Extraordinary repairs	999.38
Office expenses, superintendence, surveys, etc.	5,959.02
Total cost of dredging	$54,927.19

This dredge operated in Gulfport Harbor, Miss., from July 1 to December 10, leaving for Havana, Cuba, on December 17.

OPERATION OF DREDGES.

This paper can not be closed without reference to the operation of dredges in general, and to spud type pipe line dredges in particular. No matter how perfectly the dredge, machinery and attendant plant are constructed, the results accomplished may be very unsatisfactory unless properly managed, operated, and repaired. Contractors' dredges have been delayed waiting for fuel, or while replacing boiler tubes only a few months old, due simply to using the salty

or brackish water in Mobile Bay rather than pay the small price necessary to have fresh water brought to them; still others have been delayed on account of the breaking of a spud, no extra ones having been provided. The above are only a few of the causes for delays that have been noted. In addition the moral effect on the crews in such cases is a large incident in the mismanagement of the dredge, as the difference between the crew on a well managed, well kept dredge and that on one that is the reverse, where a breakdown is expected at any time, is very pronounced. It is believed that the plant should be kept in thorough repair and in first-class working condition at all times, instead of running it as long as possible—perhaps a year or more—as some contractors do, before giving it a general overhauling.

It is believed that time and money are saved by keeping the dredge in thorough repair at all times, briefly for the following reasons:

(a) Those parts that are liable to break and cause delays while the dredge is operating are looked after.

(b) It is cheaper to repair at the instant when needed rather than put it off until considerable wear has occurred or perhaps a breakdown causing probable damage to other parts.

(c) The spirit of the crew, which adds largely to the efficiency of the plant, is greatly benefited by keeping the plant in first-class condition at all times.

A full extra set of those parts that are liable to break and cause a delay should always be on hand. Of course, there may be some objection to having too many duplicate parts lying around, but since they do not deteriorate and have to be bought sooner or later, they had better be bought in the first place and kept out of the way in some warehouse. It takes time to furnish some of these extra parts, and as the cost of the dredge to the United States is over $300 per working day, this amount would have to be paid anyhow and might be exceeded two or three times by the loss of a single day's work, if based on contractors' prices. The dredges and their attendant plant in this district are docked twice a year for a general overhauling, and for painting below the water line, and all repairs that can not be made while the dredge is operating are attended to at this time. These two overhaulings consume about one month out of the year. During this time only such work is given to the ship yards and shops as can not be done by the

regular crew of the dredge in the required time—that is, within the time needed for the general overhauling.

In addition to the above time used in overhauling the plant, twelve hours are consumed each week in washing and scaling boilers and repairing or overhauling such parts as need it in order to keep them in good condition. The dredge starts work at 12 o'clock Sunday night and works continuously, unless interrupted, until 12 o'clock, noon, Saturday. This means that twenty-six working days are consumed during a year in this way, which is practically a calendar month, making about two calendar months consumed in overhauling and repairing the plant during the year. But during the rest of the time the dredge is ready for work, and the crew is anxious to show what can be done. Contractors in this district work continuously from 12 o'clock Sunday night till 12 o'clock Saturday night, but their plant is never in good condition, and sooner or later they have a large repair bill with a long period of lost time, besides having a plant that has greatly depreciated in value and efficiency. The Government dredge, on the other hand, is kept in a state of best efficiency at all times, while during a portion of the time the contractor's dredge is not working nearly up to her standard. As a result of the twelve hours per week devoted to the upkeep of the plant all machinery and boilers on the Government dredges in this district are in excellent condition, and no time is lost on account of repairs to them. Each man in the crew is in excellent spirits and anxious to work at all times, not as a mere machine, but as an active, eager, interested element on a large work, each feeling that something depends on him and taking a personal pride in the output of the dredge. The usual work of overhauling and greasing cables, washing boilers, shifting pipe line, overhauling machinery and other things that can not be attended to while the dredge is operating is attended to by the regular watch on duty Sunday. The men on this watch, of course, alternate each Sunday with another watch. Three shifts are made per twenty-four hours in the crews in the engine, boiler, and lever rooms, each man working eight hours, while only two shifts are made in the deck crew.

The crews on the dredges *Pascagoula* and *Wahalak* are about the same as those on similar dredges belonging to contractors, and consist generally of about forty men, as shown on opposite page.

Masters	1
Stewards	1
Lever men	3
Deck hands	15
Chief engineers	1
Assistant steam engineers or machinists	3
Oilers	4
Stokers	6
Carpenters	1
Blacksmiths	1
Cooks	1
Waiters	3
Total	40

In addition to the above, the crew of the attendant tug boat consists of eight men, and of the launch three men.

The subsistence in the raw state costs from \$12 to \$15 per man per month.

One of the most difficult and important duties connected with the supervision of dredging operations is the selection of an efficient crew. *Excellent* results can be obtained with an *ordinary* dredge, and *less* than *ordinary* results from an *excellent* dredge, depending on the crew employed. The most important positions to fill are those of master, chief engineer, and lever men. It has been noticed that a change of masters alone resulted in an increase of 50 to 75 per cent of the yardage dredged. The master is responsible for the dredge and for its proper operation, and each department is subject to his orders. He and the chief engineer generally select their own deck hands, oilers, stokers, and any others not in the Civil Service, unless the position to be filled is directly in line of promotion to the higher grades. Any failure of the master to select the proper men or maintain discipline or infuse the proper spirit in his crew or to familiarize himself with all the workings of the dredge at all times, or to set an example for forethought and industry will soon become noticeable in the results accomplished.

But no matter how able is the master, he can do little without the assistance of a capable, experienced, energetic chief engineer. The chief engineer has charge of the machinery and boilers and all repairs, machine shop and blacksmith shop work, and on one of the modern dredges he must be an expert. Considerable sums of money and much valuable time, consumed in repairs, can be saved and many breakdowns avoided by watching the *little* things about

the plant, in addition to which a great many improvements can generally be made here and there. The chief engineer must be constantly on the alert, and must know how to handle the engineers, machinists, blacksmith, oilers, and stokers under him in order to get even fair results.

Next in importance to the master and chief engineer are the lever men. In the absence of the master they are in charge of the dredge, but their principal duty is to see that the greatest efficiency is obtained in dredging material. They are not only responsible, under the master, for the quantity of material dredged, but for the position and condition of the cut formed.

Having the three positions above named filled with competent, efficient men, it becomes comparatively easy to fill the other positions; for, with those three as a nucleus, a good crew can be built up; but, and here the trouble comes in, able masters, energetic chief engineers, and efficient lever men are so scarce that fortunate, indeed, is the contractor or district officer who has these three positions well filled.

Accidents and Damages to Vessels on the Great Lakes and Connecting Channels, 1901-1910

BY

Lieut. Col. JOHN MILLIS
Corps of Engineers; Member American Society of Civil Engineers

The tables of data contained in this article are based on information derived entirely from the official reports of the United States Steamboat Inspection Service, and from those of the Department of Marine and Fisheries of Canada. Without going into a detailed discussion of the statistics, it may be well to refer briefly to some of the general results shown by the tables.

The localities are arranged as follows, according to number of accidents that were reported during the ten-year period:

	Number
Lake Michigan	888
Lake Erie and Niagara River	614
Lake Superior	371
Detroit River	269
Waterway, Lake Superior to Lake Huron	245
Waterway, Lake Huron to and including Lake St Clair	223
St. Lawrence River	172
Lake Huron and Georgian Bay	165
Straits of Mackinac	53
Lake Ontario	31
Total	3,031

According to totals of amount of damages, these localities take the following order:

	Amount
Lake Superior	$5,108,322
Lake Michigan	2,944,148
Lake Erie and Niagara River	1,990,832
Lake Huron and Georgian Bay	1,796,421
Waterway, Lake Huron to and including Lake St. Clair	1,100,461
Waterway, Lake Superior to Lake Huron	1,031,570
Detroit River	710,643
St. Lawrence River	259,052
Straits of Mackinac	206,900
Lake Ontario	131,846
Total	$15,280,195

While Lake Michigan, with its connecting harbors and bays, shows the greatest number of accidents, those of Lake Superior involve the greatest total damage. Of the narrow channels and waterways Detroit River heads the list in number, and the St. Clair River and canal in amount of damage. At Whitefish Point and Bay, the amount of loss relative to number of accidents is greater than at any other locality. Certain harbors show a very large number of accidents by collision, but the amount of damage in these localities from this cause is generally small, as would naturally be expected.

By far the largest number of accidents resulted from the collision of vessels, each collision between two vessels being counted as two accidents, and the classification showing the greatest total of damages is ''strandings and striking submerged obstructions.'' These two classifications concern more directly the work of the Engineer Department on the lakes than any of the others, and the importance of devising ways and means of reducing these classes of accidents is apparent.

A practical measure of the present risks of navigation on the lakes is afforded by the cost of insurance. For a 10,000-ton ore vessel, costing about $300,000 when new, the annual insurance premium is now $19,250. It was the very large insurance charges that led primarily to the organization of the Great Lakes Protective Association, and this Association is making a careful study of the general subject of the risks of navigation on the lakes. A similar organization has been formed by Canadian vessel owners.

A branch of the subject that calls for very careful consideration is the general efficiency of the vessel itself; its design and equipment, the skill and competency of the master and other officers, sufficiency of the crew, adequacy of the appliances for providing safety to life, regulation of the loading, etc. These are more likely to receive proper attention from the vessel owners than some of the more general features of the question external to the vessel, and the former were not therefore made the subject of special study in compiling this report.

During the ten-year period here considered the amount of commerce on the lakes, as measured by the total freight handled, has more than doubled, and so many other different elements are involved that it is not practicable to deduce any very direct relation between the number of accidents and amount of damages, and the

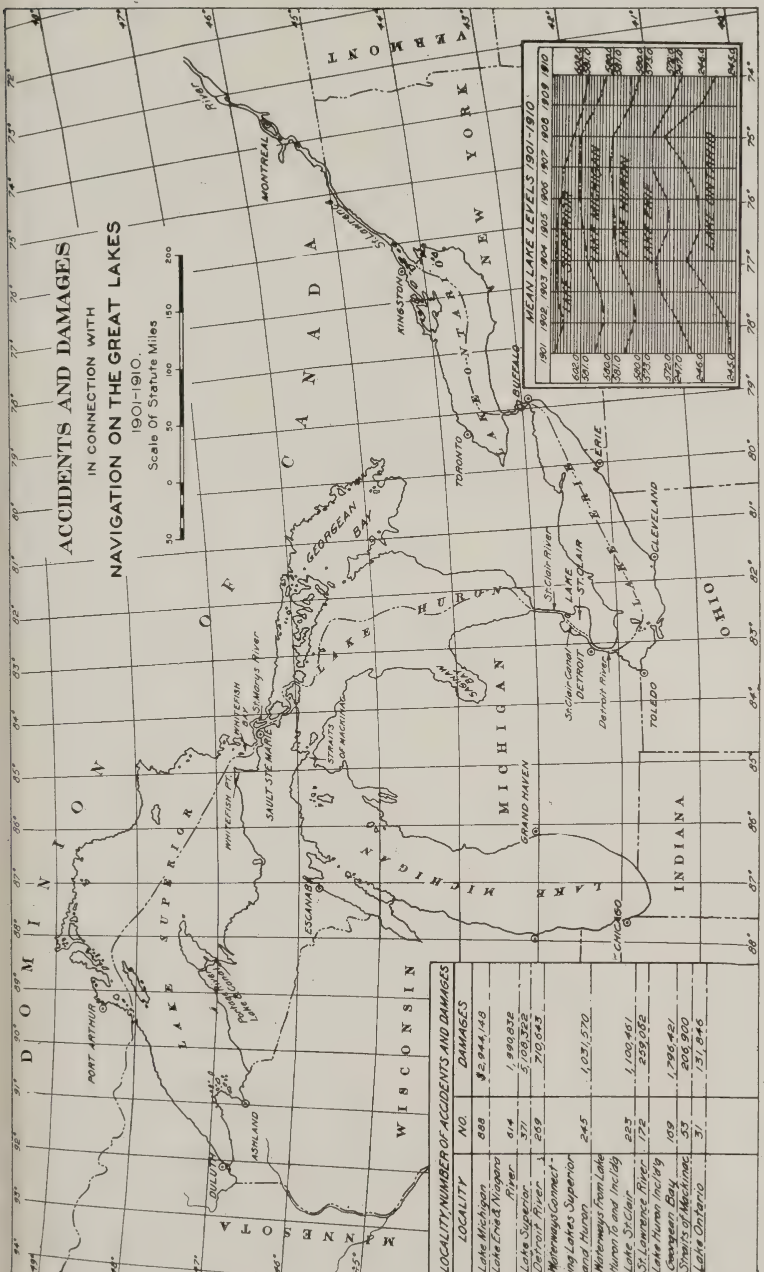

ACCIDENTS AND DAMAGES
IN CONNECTION WITH
NAVIGATION ON THE GREAT LAKES
1901-1910.
Scale Of Statute Miles
MEAN LAKE LEVELS 1901-1910
LAKE SUPERIOR
LAKE MICHIGAN
LAKE HURON
LAKE ERIE
LAKE ONTARIO
LOCALITY, NUMBER OF ACCIDENTS AND DAMAGES
LOCALITY NO. DAMAGES
Lake Michigan 888 $2,944,148
Lake Erie & Niagara River 614 1,990,832
Lake Superior 371 5,108,322
Detroit River 259 710,643
Waterways Connecting Lakes Superior and Huron 245 1,031,570
Waterways from Lake Huron to and Incl'dg Lake St. Clair 225 1,100,461
St. Lawrence River 172 259,062
Lake Huron Incl'dg Georgean Bay 109 1,796,421
Straits of Mackinac 53 206,900
Lake Ontario 31 131,846
DOMINION OF CANADA
NEW YORK
VERMONT
MICHIGAN
WISCONSIN
MINNESOTA
INDIANA
OHIO

Plate I.

works of channel and harbor improvement that have been carried out during the period considered.

The most important channel improvements affecting the main waterways that have been made since 1900 are as follows: In the waterway between Lake Superior and Lake Huron, the opening of the new West Neebish Channel, having a minimum width of 300 feet and a minimum depth of 21 feet; the removal of numerous shoals and rock ledges, beginning the construction of a new lock and approach channels at Sault Ste. Marie, and improvements to the approaches to the Poe Lock. At the St. Clair Flats Canal, in 1900, there was one channel 292 feet wide and 20 feet deep, but now there are two channels—one 292 feet wide and 20 feet deep, the second 300 feet wide and 20 feet deep. In the Detroit River, in 1900, there was one channel ranging in width from 300 to 600 feet and in depth from 17½ to 21 feet. At the present date there is an available channel from 600 to 800 feet in width and 19½ to 21 feet deep, with work well advanced on a project which will provide one channel 600 to 800 feet wide and 21 feet deep from Ballards Reef to Lake Erie, and a second channel 300 to 800 feet wide and 22 feet deep (the Livingstone Channel) between the same points. A large amount of similar improvement work has been done in the past ten years on less important channels, including the St. Lawrence River, and much has also been done to improve the depths and capacity of harbors and the safety of their entrances, both by the Canadian and the United States governments.

Attention is invited to curves on general map, showing changes in mean lake levels on the several lakes during the ten-year period. On all the lakes the mean elevations have been diminishing during the last three to four years of the period, and during the preceding years they were increasing or stationary, while the usual seasonal variations have, of course, occurred. With the attention now being given to water-power development, this question of levels is becoming one of the most important ones in connection with lake navigation.

It should be added that there is room for improvement in the accuracy and completeness of the data furnished by the official reports. There is good reason to believe that many accidents are not included in these reports, and that the estimates of damages are often unreliable.

The following is an extract for a paper for the coming Navigation Congress.

A study of the available data summarized in the foregoing has

SUMMARIES

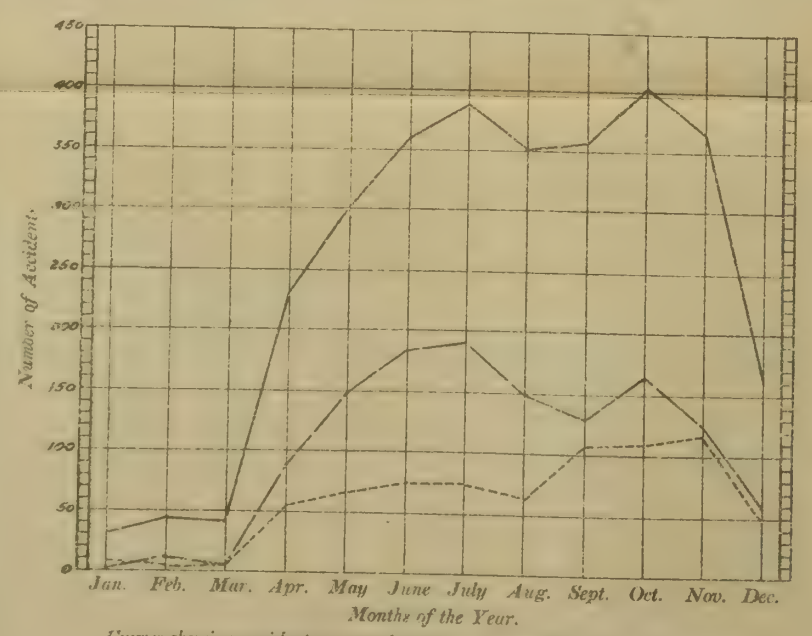
1901-1910 — CALENDAR YEARS — 1901-1910

SUMMARIES

1901-1910 — CALENDAR YEARS — 1901-1910

LAKE or RIVER	LOCALITY	ACCIDENTS AND AMOUNT OF DAMAGES															
		Collisions between Vessels (a)		Collisions with docks or fixed Obstructions not Submerged		Stranding or Striking Submerged Obstructions		Accidents to Machinery or Equipment		Fires		Sunk or (b) Damaged by Stress of Weather		Unclassified Accidents		Totals	
		No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount
Lake Superior	Duluth	60	$400,139	14	$139,900	12	$70,165	5	$2,350	13	$43,150		$	6	$850	110	$706,474
	Two Harbors	6	4,750	4	6,800	1	1,500	1	12,000							12	25,050
	Ashland	2	2,000							2	11,000			1	500	5	13,500
	Marquette	2	——			1	1,000	4	4,350	1	10,000					8	15,350
	Other Harbors	2	800			4	45,150	5	3,025	4	50,550	3	1,575			18	101,100
	Whitefish Pt. & Bay	22	401,250			6	40,800	5	3,200	2	1,450	5	292,000			40	738,700
	Portage L., R. & Canal	2	1,800	1	500	7	19,100	1	3,600	1	45,000	1	4,000	1	1,500	14	75,500
	Vicinity Apostle Isl.					18	494,335	1	——	1	1,900					20	496,235
	Islands and Reefs	6	38,000			39	836,095	3	17,150			1	900	2	67,000	51	960,045
	Other Localities	2	2,000	1	1,000	19	475,950	4	3,700	1	1,000					27	483,650
	Open Lake	10	175,250			4	55,600	21	104,118	8	115,000	24	966,150	5	76,600	66	1,492,718
Totals		114	1,086,389	20	198,100	111	2,039,715	50	154,293	27	278,850	34	1,264,625	15	146,450	371	5,108,322
Waterways Connecting Lakes Superior and Huron	Sault Ste. Marie	61	86,720	18	207,125	13	36,900	3	4,315	2	22,000					97	357,060
	Sailors Encampment					8	25,100									8	25,100
	Mud & Hay Lakes	8	5,400			5	66,300									13	71,700
	St. Mary's River	75	356,450			37	179,450	11	6,310	2	31,000	2	4,500			127	577,710
Totals		144	448,570	18	207,125	63	307,750	14	10,625	4	53,000	2	4,500			245	1,031,570
Lake Huron Including Georgian Bay	Georgian Bay					6	11,000	3	8,350	5	57,000			1	350	15	76,700
	Saginaw Bay							1	500	4	24,750			3	56,000	8	81,250
	Islands and Reefs	2	500			25	148,046	2	5,300	1	8,000					30	161,846
	Other Localities			3	8,000	18	101,650	2	750	8	32,850					31	143,250
	Open Lake	40	654,925			10	142,300	14	6,150	7	185,000	2	1,500	8	346,500	81	1,335,375
Totals		42	655,425	3	8,000	53	402,996	22	19,050	25	307,600	2	1,500	12	401,850	165	1,796,421
Lake Michigan	Milwaukee	65	35,610	18	6,390	15	138,900	12	3,430	6	15,160			2	200	118	199,690
	Manistique	4	475			4	14,345	2	525	4	1,750			1	10,000	15	27,095
	Manitowoc	8	1,300	1	2,000	2	1,100	7	3,500	2	114,800			1	1,500	21	124,200
	Sheboygan	6	1,825	1	200	1	——			1	100			1	300	10	2,425
	Chicago	154	20,830	65	58,421	14	23,200	28	23,090	24	64,240	1	200	10	7,365	316	217,436
	Grand Haven	6	1,437	1	——	2	275	4	960	3	10,250			1	3,500	17	16,422
	Ludington	4	600			2	50,600	4	52,200							10	103,400
	Other Harbors	36	5,175	11	11,477	28	116,968	15	5,823	21	88,354	1	5,000	6	10,610	108	244,407
	Green Bay	4	12,100			7	5,200	3	1,150	5	115,225					19	133,675
	Manitou Islands	4	60			7	272,279			1	45,000					12	317,339
	Sturgeon B. & Can.	4	100			3	500	2	7,500	1	300			1	300	11	8,700
	Islands and Reefs					48	462,800	1	——	1	2,000	1	50			45	464,850
	Other Localities	6	300	2	200	30	178,200	1	100	7	43,070	2	2,515	1	1,500	49	225,885
	Open Lake	24	16,985			18	65,250	67	50,289	9	504,300	11	320,000	8	91,800	137	958,624
Totals		315	96,887	119	78,688	175	1,348,617	146	148,557	95	935,549	16	327,765	32	117,075	898	2,944,148
Straits of Mackinac	Cheboygan					3	——	1	400	3	11,500					7	11,900
	St. Ignace					1	——	1	350	1	2,500					3	2,850
	Straits of Mackinac	10	2,500			28	90,600	7	6,050	1	50,000	2	43,000			48	192,150
Totals		10	2,500			32	90,600	9	6,800	5	64,000	2	43,000			58	206,900
Waterways from Lake Huron to and Including Lake St. Clair	Lake St. Clair	32	39,100			8	15,690	4	10,805			1	——	1	2,500	46	68,095
	St. Clair River	116	584,400	2	6,750	10	60,250	4	2,380	8	193,100	1	4,000			141	850,880
	St. Clair Canal	16	50,749	1	3,000	3	——							1	1,000	19	54,749
	Docks & Landings	4	12,100	1	3,000	1	300			11	105,337					17	120,737
Totals		166	685,349	4	12,750	22	77,840	8	12,585	19	304,437	2	4,000	2	3,500	223	1,100,461
Detroit River	Detroit	8	2,175	5	2,865			1	——	7	26,500			3	6,000	24	37,540
	Detroit River	122	334,619			36	117,100	16	12,639	5	26,800					179	491,158
	Ballard's Reef	9	11,000			13	48,050									22	59,050
	Limekiln Crossing	12	21,084			15	97,951	2	3,650							29	122,685
	Bar Point	6	90			8	——	1	120							15	210
Totals		157	368,969	5	2,865	72	263,101	20	16,409	12	53,800			3	6,000	269	[illegible]
Lake Erie and Niagara River	Toledo	21	6,433	5	36,938	4	5,000	4	3,950	10	41,600	2	11,000	2	200	48	105,116
	Sandusky	4	775	2	1,200	15	23,290	3	275	2	2,000			1	1,000	27	28,540
	Lorain	4	700	4	7,500	5	10,100	2	——	2	16,500			2	23,000	19	57,800
	Cleveland	48	5,925	6	3,500	5	4,800	3	2,000	9	15,375	2	8,250	2	——	75	39,850
	Fairport	2	——	1	3,000	2	8,000									5	11,000
	Ashtabula	13	3,950	7	20,500	7	32,500							2	500	29	57,450
	Conneaut			3	5,000	2	——			2	500					7	5,500
	Erie	4	12,500	3	4,500	1	500	1	600	7	12,200					16	30,300
	Dunkirk					3	6,130			2	4,100					5	10,250
	Buffalo	99	51,775	21	30,225	21	40,200	6	1,800	17	66,100	1	25	7	——	172	190,125
	Other Harbors			1	1,600	4	7,000			3	17,125					8	25,725
	Islands and Reefs	18	39,075			26	74,550	2	250	2	4,700	1	25,000	1	——	50	143,575
	Niagara River	2	——			15	80,600	5	2,375	5	53,750			2	270	29	136,995
	Other Localities	2	——			4	4,000	2	5,100	2	20,400					10	29,500
	Open Lake	24	30,200			10	32,500	42	158,326	12	307,500	24	689,600	2	1,000	114	1,119,126
Totals		241	151,333	53	113,958	124	329,170	70	174,676	75	461,950	30	733,875	21	25,970	614	1,990,832
Lake Ontario	Totals	4	10,000			8	25,500	7	810	7	58,600	3	3,736	2	33,200	31	131,846
St. Lawrence Riv.	Totals	55	31,785	7	2,300	64	144,016	21	3,553	19	62,349	2	1,050	4	14,000	172	259,052
Grand Totals		1248	3,477,206	229	623,786	725	5,030,305	367	547,368	279	2,469,534	93	2,383,951	91	748,045	3031	(c) 15,280,195

(a) Each collision between two vessels is counted as two accidents.
(b) This column includes damages due to stress of weather. These are not included under the other headings.
(c) Amount of damages approximate, information not always available.



Curves showing accidents grouped according to month in which they occurred.
Total number of accidents. ————
Accidents due to collisions between vessels. — — — —
Accidents due to stranding etc. - - - - - -

U. S. ENGINEER OFFICE
CLEVELAND, OHIO
September 6, 1911

Sheet 11

J. Millis
Colonel Engineers

led to the following suggestions relative to improvements, etc., that seem worthy of consideration with a view to reducing the number of accidents and the risks of navigation on the Great American Lakes:

A closer and more systematic cooperation and understanding between the officials of the Canadian government and those of the United States Government in all matters concerned with the navigation of the lakes.

The extension, as far as may be practicable, of the separate up-bound and down-bound channels and available courses for the waterway from outside Whitefish Point, Lake Superior, through St. Marys River and other channels to Lake Huron, and for that from Lake Huron to Lake Erie, especially in Detroit River, with necessary supervision to regulate the movement of vessels in these channels. Much has already been accomplished in this respect, and more is now in progress.

Official establishment of separate courses for up-bound and down-bound vessels in the open lakes where the greatest danger of collision exists, and proper action to enforce observance of such courses and established "rules of the road" under government supervision. Such a system has already been inaugurated on Lake Huron by the Great Lakes Protective Association for the vessels of its members.

Further study of the problem of maintaining the levels of the lakes and regulating the variations of levels to meet the requirements of navigation, in connection with questions of utilizing all sites practically available for water-power development.

Improvements in sound signalling apparatus, on the vessels as well as on shore, with special reference to greater certainty that such signals will be heard and correctly interpreted by masters and pilots of vessels.

More effective action to regulate or prevent navigation during unsafe weather conditions at the beginning and close of the usual season of navigation.

Improvement in the means of handling the larger vessels in narrow and crooked channels and in harbors.

A more practical basis of coordination between artificial harbors and channels—including locks—and the vessels that are to use them, both in dimensions and in phases of development of the vessels and of the channels, etc., respectively.

Such legislation or other action as may be necessary to enable the proper government officials to promptly investigate all cases of accident or damages to vessels, to ascertain correctly responsibility, cost of repairs and other details, and to arrive at and report conclusions without delay. Such inquiry and report to be made independent of litigation in the courts, which is often a matter of years.

The creation of an international board, commission, or other tribunal, to deal as above with cases of collision or other accidents involving vessels of both nationalities.

It is further suggested that as the cause of accidents to vessels is frequently a matter of interest in connection with the work or the administration service of several different departments of the Government, the investigation of accidents should be by a board composed of representatives of these several departments, or it might be by an official or officials independent of all of them.

The writer desires to acknowledge the assistance received from the several officials of the Canadian government who were consulted in the preparation of this article, and particularly the courtesies of Hon. Alexander Johnston, Deputy Minister of Marine and Fisheries, of Ottawa.

SUMMARIES
1901 - 1910 — CALENDAR YEARS — 1901 - 1910

Table 1:- Accidents and damages yearly 1901 - 1910, inclusive, grouped according to nature of accident.

Year	Collisions between Vessels		Collisions with docks or fixed Obstructions not Submerged		Stranding or Striking Submerged Obstructions		Accidents to Machinery or Equipment		Fire		Sunk or Damaged by Stress of Weather		Unclassified Accidents		Totals	
	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount
1901	72	$ 42,575	3	$ 1,450	25	$ 84,935	11	$ 57,250	31	$ 387,575	5	$ 202,300	7	$ 95,800	154	$ 971,885
1902	89	384,525	9	10,057	39	149,185	18	169,523	23	208,725	4	12,600	8	25,150	190	960,865
1903	59	22,150	6	28,205	37	390,325	20	27,540	26	193,025	7	198,000	8	69,270	163	1,022,515
1904	59	113,344	5	4,105	36	328,330	34	16,490	36	204,390	3	1,565	10	58,525	183	726,449
1905	111	597,715	21	164,530	73	1,577,300	33	101,282	21	204,300	14	93,000	13	158,200	286	2,896,327
1906	178	535,690	34	37,177	93	446,315	42	16,588	22	232,852	10	33,450	6	121,275	385	1,413,347
1907	194	501,522	40	61,470	71	385,500	62	50,809	36	336,507	15	316,550	9	36,210	427	1,688,568
1908	168	178,909	35	86,778	118	622,351	38	29,098	33	235,350	8	342,066	10	21,750	411	1,516,338
1909	180	714,221	43	210,964	131	465,365	66	46,354	30	252,290	15	878,300	8	80,450	473	2,627,944
1910	138	196,455	38	25,050	101	590,683	43	32,444	20	234,610	12	306,100	12	80,515	359	1,455,957
Totals	1248	3,477,205	229	623,786	725	5,030,305	367	547,268	278	2,459,534	93	2,383,931	91	745,045	3031	15,280,195

Table 2:- Accidents and damages yearly 1901 - 1910, inclusive, grouped according to locality.

Year	Lake Superior		Waterways connecting Lakes Superior and Huron		Lake Huron including Georgian Bay		Lake Michigan		Straits of Mackinac		Waterways from Lake Huron to & including L. St. Clair		Detroit River		Lake Erie and Niagara River		Lake Ontario		St. Lawrence River		Totals	
	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount	No.	Amount
1901	21	$ 374,900	7	$ 52,000	11	$ 65,300	44	$ 230,575	2	$ 10,000	22	$ 64,150	13	$ 35,000	28	$ 39,550	1	$ -----	5	$ [illegible]	154	$ 971,885
1902	32	357,705	11	18,200	12	127,450	34	114,382	2	17,750	7	31,500	15	27,200	43	223,775	2	1,400	32	[illegible]	190	960,885
1903	20	395,700	5	31,400	6	87,800	44	191,680			17	103,500	16	80,100	39	127,825	2	3,000	12	11,500	163	1,022,515
1904	16	99,259	7	25,500	4	128,805	71	244,710			20	74,350	5	53,000	30	48,100	5	41,075	13	5,550	183	726,449
1905	54	1,585,175	16	19,200	21	464,800	87	454,967	1	30,000	24	[illegible]	23	53,900	36	95,500	3	300	21	30,485	286	2,838,327
1906	44	182,735	44	234,300	24	332,200	119	387,419	7	20,300	29	148,300	25	63,665	77	89,480	1	2,000	20	18,948	385	1,413,347
1907	54	437,350	42	100,630	12	71,400	138	506,609	16	29,050	27	251,387	47	245,177	77	177,600	2	100	8	9,365	427	1,688,568
1908	44	570,053	34	86,415	20	253,366	119	140,519	13	4,100	24	78,343	43	38,096	88	224,773	6	16,086	21	[illegible]	411	1,516,338
1909	55	889,570	47	387,325	22	182,100	115	157,089	14	74,850	36	106,533	31	86,555	113	721,425	3	95	17	[illegible]	473	2,627,944
1910	31	174,875	32	76,600	24	98,200	122	766,198	4	900	17	63,990	32	35,950	83	158,794	8	67,600	5	12,950	358	1,455,957
Totals	371	5,108,322	245	1,031,570	165	1,796,421	893	2,944,149	58	206,900	223	1,100,461	258	710,643	614	1,990,832	31	131,846	172	258,052	3031	15,280,195

Table 3:- Accidents yearly 1901 - 1910, inclusive, grouped by months.

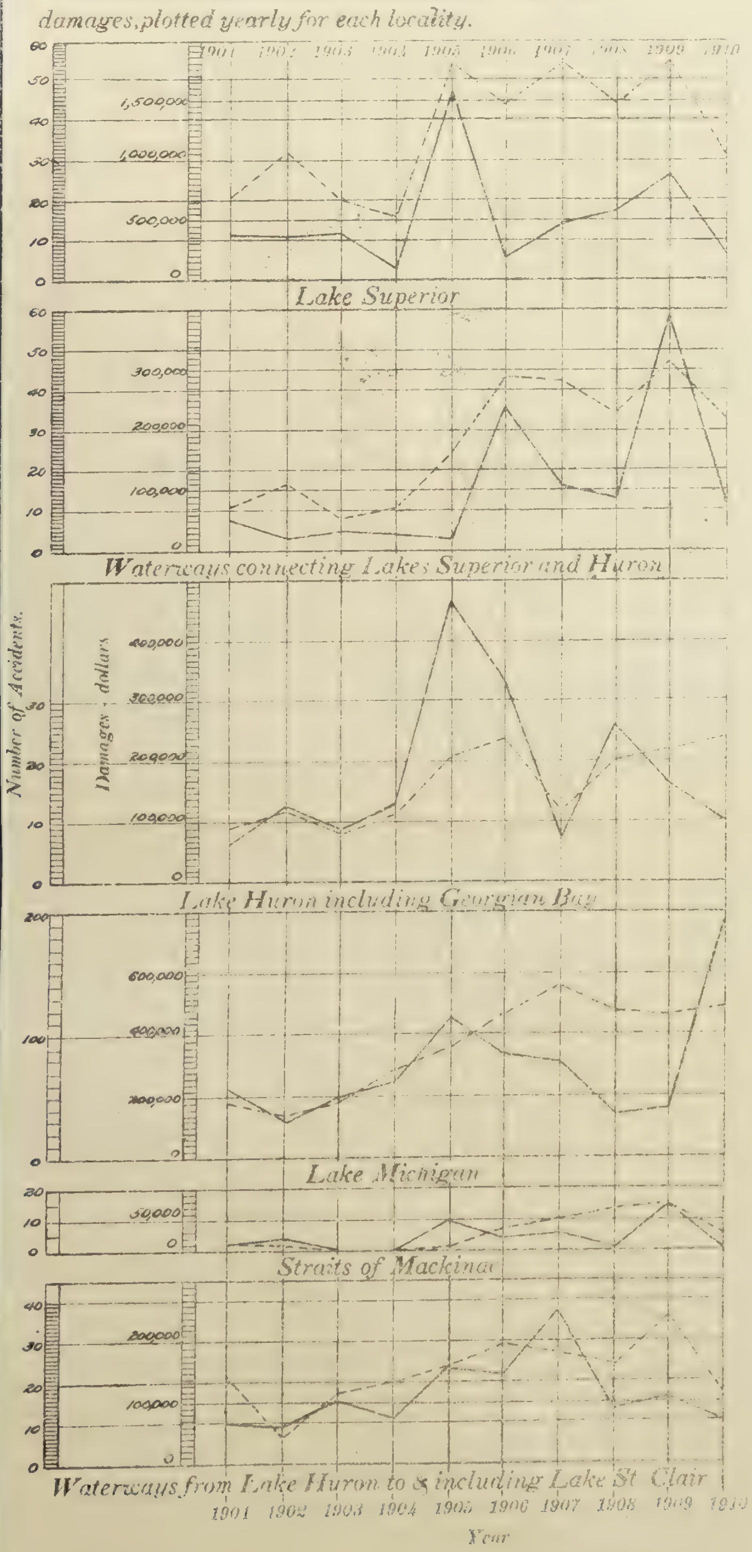
Year	Jan	Feb.	Mar	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec	Totals
1901	3	1	2	2	24	19	28	20	12	25	14	9	150
1902	1			13	22	25	25	25	26	11	20	3	190
1903	1	2	1	7	14	18	38	23	13	23	25	3	165
1904	3	10	9	9	12	27	35	16	9	20	22	12	183
1905	1	6	4	21	31	40	21	26	32	35	57	12	286
1906	5	4	3	25	33	58	51	55	44	52	33	22	385
1907	6		1	59	49	38	42	52	62	61	37	21	427
1908	3	5	8	11	27	41	58	39	69	62	55	33	411
1909	4	5	4	36	45	45	48	63	55	72	68	27	473
1910	3	9	8	44	46	41	50	32	35	41	34	16	359
Totals	30	42	40	232	302	359	387	351	356	403	365	164	3,031

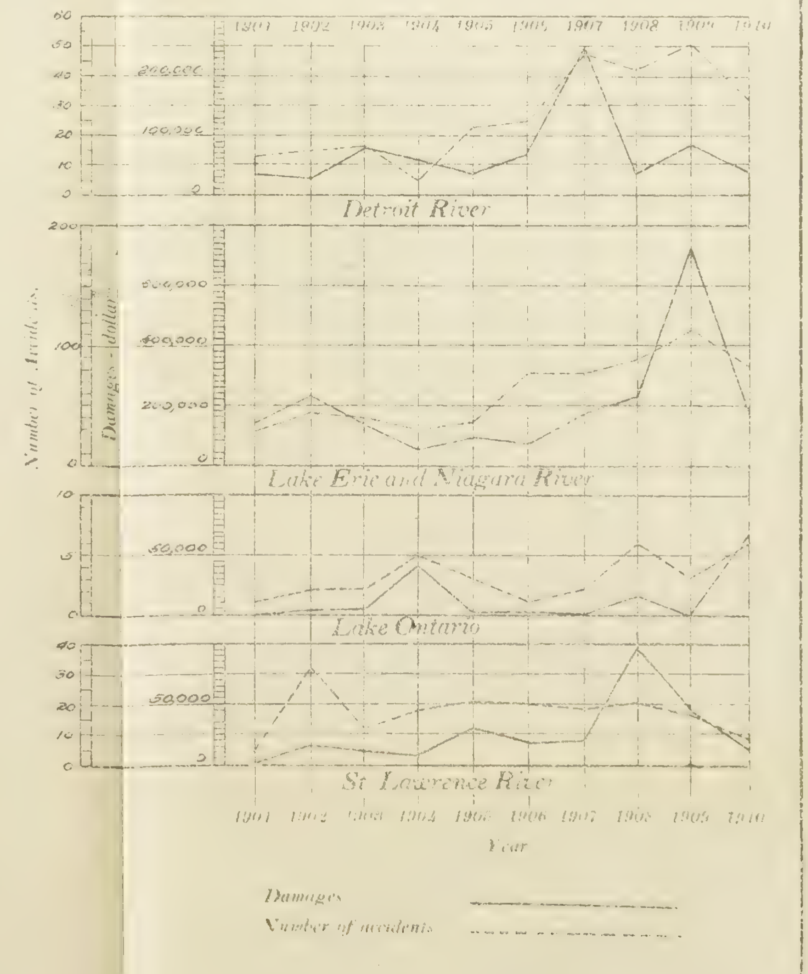
Table 4. Ten accidents during period 1901 - 1910, inclusive in which the greatest damages were sustained

Lake Superior	Stress of weather	Nov. 1905	$ 330,000
Lake Michigan	Stress of weather	Sept. 1910	300,000
Lake Superior	Stranded	Nov. 1905	300,000
Lake Huron	Collision between vessels	June 1905	287,000
Lake Superior	Stress of weather	Oct. 1907	280,000
Lake Superior	Collision between vessels	June 190[illegible]	225,000
Lake Erie	Stress of weather	Dec. 1909	224,000
Lake Superior	Stress of weather	Apr. 1909	200,000
Lake Michigan	Stranded	May 1910	200,000
Lake Superior	Stress of weather	Sept. 1910	180,000

Note :- In November 1905 one storm on Lake Superior caused nineteen accidents with total damage of $1,005,800

Curves showing number of accidents and resulting damages, plotted yearly for each locality.





U. S. ENGINEER OFFICE
CLEVELAND, OHIO
September 6, 1911

Sheet 12

J. Millis
Colonel Engineers

Regulation of the Hiwassee River near Charleston, Tenn.

BY

Mr. NICHOLLS W. BOWDEN
Junior Engineer

Though this paper is to deal primarily with the various shoals on the Hiwassee River below Charleston, Tenn., their description, condition before and after improvement, location, etc., it would in my opinion be incomplete were it not to contain, as an introduction, a general description of the river and the country through which it flows.

The section of the country contiguous to the Hiwassee River between Charleston and its confluence with the Tennessee River, 19 miles below, is mountainous, yet not nearly so mountainous as that above Charleston. Numerous ridges are cut through by the river in its northwest course. These ridges are generally at right angles to it and parallel to the Tennessee River. A feature that might well be mentioned here is that the river is comparatively straight; a straight line from Charleston to its mouth measures 17 miles, while the distance by river measures but 19 miles. From the valleys several large and many small creeks empty into the river, increasing the low water discharge from about 1,000 cubic feet per second at Charleston to 1,200 cubic feet per second at its mouth. The average height of bank along the river is not more than 20 feet, high water often inundating large areas of cultivated land.

From the above description of the river basin we would naturally expect to find a river with a narrow bed, a great axial slope, and a swift current. What was actually found before extensive improvements were made was a river whose bed was narrow (varying from 275 feet at Charleston to 425 feet nearer its mouth, averaging possibly 350 feet); whose mean fall was small, being less than 0.8 foot to the mile, but concentrated at the shoals, and whose current velocity was excessive at several localities. The above applied in 1899, before which time only meager improve-

ments had been made. Referring to the statement above that the fall was concentrated at the shoals, it can be said that of the total fall of 15 feet, between Charleston and its mouth, 11 feet or nearly 75 per cent of the fall is taken up at various shoals which cover only 4.2 miles, or 22 per cent of the entire distance.

At the present time (January 1, 1911) we find quite a different river, one in which the slope is gradual even on the shoals, where a maximum slope of 18 feet to the mile has been reduced to 9 feet, and where excessive velocities have been reduced to a maximum of 4.25 feet per second. It is not known what the maximum velocity was previous to the works of improvement, but it is known that boats plying the river at such times could not reach Charleston without using warps at several places. At the present time boats towing heavily loaded barges have no trouble in steaming the swiftest places.

Numerous methods of improvement have been employed since 1876, when the United States first recognized this river as worthy of improvement, some of which were by the use of wing dams, training walls, spur dikes, and submerged sills, all of which were built of riprap stone. Many of these earlier methods or projects proved to be failures or to be ineffectual, either because they were wrongly placed or because they were not suited to the shoals at which they were used. To sustain this statement it is only necessary to point out Matthews Shoals, where numerous wing dams and submerged sills were placed in the river only to be taken out later and be replaced by others. All of these finally gave way to a system of longitudinal dikes and check or cross dams, which system has proven most satisfactory. This system referred to consists in contracting the channel to 150 feet and confining it to one bank by building a wing dam or training dike from one bank to connect with a longitudinal dam, located about 150 feet from the opposite bank. This longitudinal dam itself is connected at intervals with the bank by check or cross dams. These check or cross dams are built higher than the longitudinal dam in order to catch the water that leaks through above and in turn divert this water into the regulated channel, thus checking the current velocity as well as furnishing more water to the channel. It is clearly seen that the leakage of these check or cross dams is just equal to the leakage of one check, since that leakage is immediately available to supply the leakage of the next. It is also plain that were these cross dams

not built the leakage of the longitudinal dam would be cumulative and would be suppled from the channel.

At most of shoals on the Hiwassee River where this method of im-

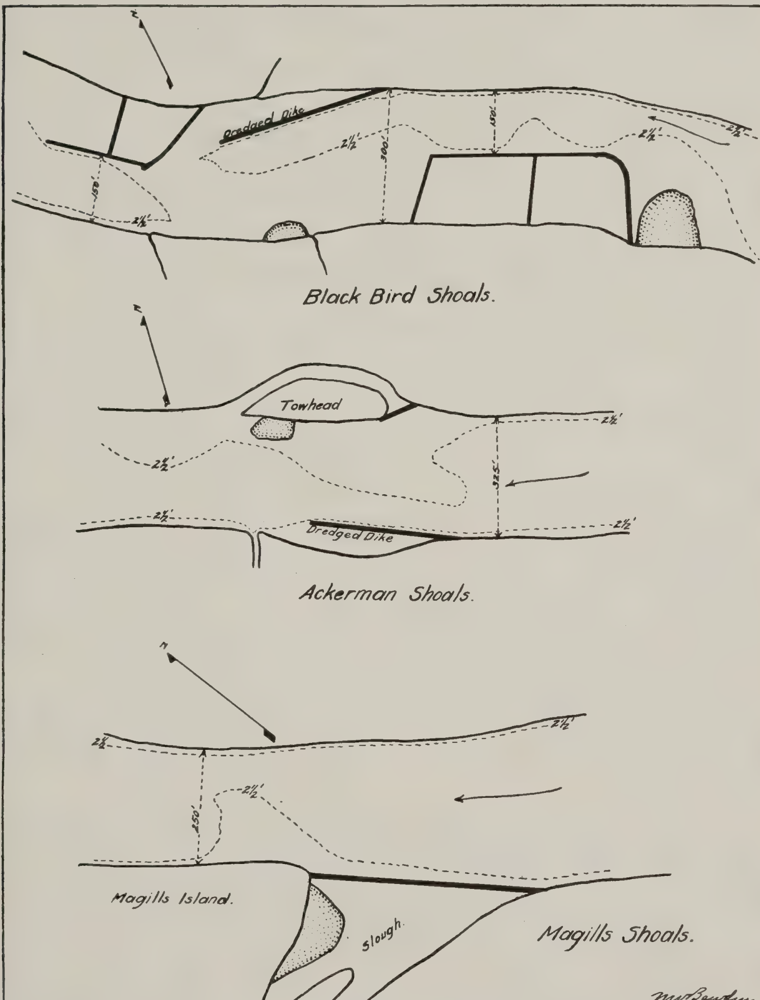


Plate I.

provement has been followed, the upper end of the longitudinal dam is connected with the bank by a heavy wing dam, which is built curved at its connection with the longitudinal dam. This curve serves to contract the channel slowly and thus prevent the

cross-current and roughness of the water that would appear at this point were the connection made angular. At several shoals this angular connection was made only to be taken out later and to have a curved connection substituted therefor. At Matthews Shoals, previous to this change, boats with any considerable tow could only get through by the use of warps; after this change the use of warps became unnecessary.

Another method of improvement, that of dredging, has recently been employed where the nature of the material of the bottom would permit. Where this method has been employed, the dredged material has invariably been placed in a high dike (some 4 feet above low water) along the edge of the cut nearest the bank and this dike is connected to the bank at the upper end. At low water this dike is a guide to pilots, who know that it is placed at the edge of the cut. When this dredged dike angles into the river, it is connected with the bank at intervals by cross dams which prevent the scouring of the bank and cause a filling-in behind the dike. Some of the dredged material is also placed in check dams, spaced to suit the conditions, extending from the opposite bank, near or below the foot of the cut, at right angles to the current, or, better, at an angle of from 15 degrees to 30 degrees upstream. The length of these check dams depends upon the amount of contraction necessary, but none of them contract the channel to less than 150 feet. These checks serve to equalize the fall and prevent the water being drawn off at the head of the cut.

Still another method of improvement has been employed, that of removing rock reefs and boulders from the channel by drilling and blasting.

The results obtained below Charleston, as will be seen below where each shoal is taken up separately, have been good. The project which calls for a low water channel 90 feet wide and 30 inches deep, has been completed except at Black Bird and Matthews shoals, where at certain places less than this desired depth is found. This statement applies from Charleston to Bunker Hill Shoals, 4 miles above the mouth, below which point no recent surveys have been made. It is known, however, that no serious obstruction appears in this stretch. Each shoal will be taken up separately below and briefly described, the conditions before improvement being compared with those after improvement.

BLACK BIRD SHOALS. (Page 207.)

One mile below Charleston. Length obstructed, 5,000 feet; total fall, 2.1 feet; mean fall, 2.2 per mile; maximum fall previous to improvement, 18 feet per mile; maximum fall after improvement, 4 feet per mile; maximum current velocity previous to improvement, not known; maximum current velocity after improvement, 3.9 feet per second.

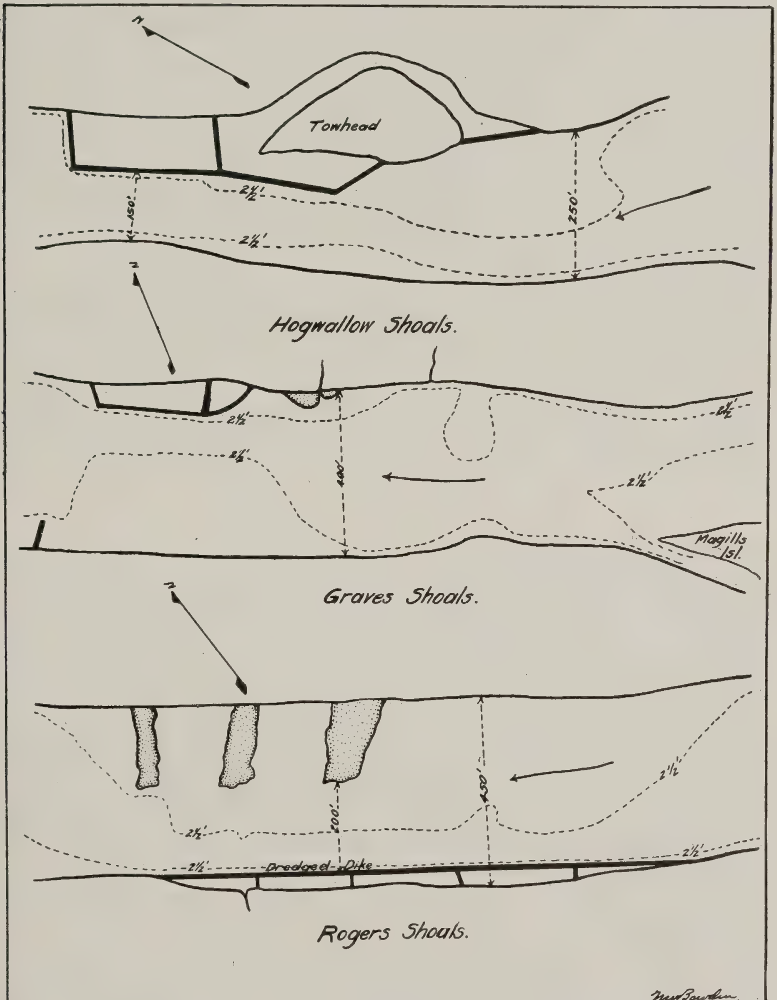


Plate II.

The bottom at these shoals is composed of solid rock, rock boulders, and gravel. These shoals were improved by two systems of longitudinal and cross dams, one at the head of the shoals 1,000 feet long and one at the foot of the shoals 800 feet long, each of which contracts the channel to 150 feet; and by dredging between these two sets of dams. The width of the dredged cut between these two sets of dams is 80 feet, and the length 1,300 feet. The dredged material, which is gravel and cemented material or hard-pan, was placed in a heavy dike along the right edge of cut and joining to the right bank at the head. The upper system of dams confine the channel to the right bank, while the lower system confines it to the left. The results here have been satisfactory, except between the lower end of the dredgd dike and the head of the lower system of dams, where, for a distance of 300 feet, the depths are insufficient. These shoals were referred to by G. T. Nelles, in his report of August 10, 1900, on Hiwassee River, as being probably the worst obstructions between Charleston and the mouth. At the present time they furnish very little trouble even at the lowest stage of water.

ACKERMAN SHOALS. (Page 207.)

Three miles below Charleston. Length obstructed, 600 feet; total fall, 0.2 foot; mean fall, 1.8 feet per mile; maximum fall and maximum current velocity previous to improvement, not known; maximum fall after improvement, 1.8 feet per mile; maximum current velocity after improvement, 2.39 feet per second.

These shoals were never a serious menace to navigation, but the depths were inadequate and it was found necessary to dredge a cut 450 feet long 90 feet wide through the gravel bar, and to close the slough at the head of a little tow-head to divert the water into the regulated channel. The material dredged from the cut was placed in a dike at the left edge of cut, which dike is joined to the left bank at the head. The results are that ample depths have been obtained.

MAGILLS SHOALS. (Page 207.)

Six miles below Charleston. These shoals are on the right chute at Magills Island, which island is about 6,500 feet long, and are generally considered as occupying the entire distance. However, here they are accepted as occupying a distance of 1,300 feet at the head of the island. All that has been done at these shoals was to build a dam across the head of the left chute to confine the water

to the right chute. The depths here are sufficient, the slope gradual, and the current velocity small.

HOGWALLOW SHOALS. (Page 209.)

Six and one-half miles below Charleston. These shoals are a part of the above-mentioned Magills Shoals.

Length obstructed, 1,400 feet; total fall, 1.4 feet; mean fall, 5.3 feet per mile; maximum fall previous to improvement, 10.6 feet per mile; maximum fall after improvement, 8 feet per mile; maximum current velocity previous to improvement, not known; maximum current velocity after improvement, 3.69 feet per second.

The improvement at these shoals was effected by closing the slough which flowed to the right of a small tow-head, and by a system of longitudinal and cross dams 650 feet long. These dams, extending from the small tow-head on the right, confine the 150-foot channel to the left bank of the river or along the right bank of Magills Island. The improvement of these shoals has been a success in that the depth and width of channel proposed has been obtained, and that the current velocity is not excessive at any point.

GRAVES SHOALS. (Page 209.)

Eight miles below Charleston. Length obstructed, 3,300 feet; total fall, 0.3 foot; mean fall, 0.5 foot per mile; maximum fall and current velocity previous to improvement, not known; maximum fall after improvement, 1 foot per mile; maximum current velocity after improvement, about 2 feet per second.

These shoals consist of a number of rock reefs extending from either bank into the river, a gravel bar at the foot of the last reef, and some large boulders in the channel. These boulders have, from time to time, been blasted from the river until a suitable channel was obtained. The reef and bar referred to have obstructed low water navigation because insufficient depths thereon. A channel has been excavated near the right bank, affording full depths. The rock and gravel taken out was placed in a dike along the right edge of cut and in a check dam extending from the left bank. This check dam is just below and opposite the foot of cut.

ROGERS SHOALS. (Page 209.)

Nine miles below Charleston. Length obstructed, 2,400 feet; total fall, 1 foot; mean fall, 2.2 feet per mile; maximum fall and current velocity previous to improvement, not known; maximum

fall after improvement, 2.7 feet per mile; maximum current velocity after improvement, 3.65 feet per second.

The improvement at these shoals consisted in dredging a channel, 90 feet wide and 1,960 feet long down the left bank, through gravel and hardpan. As at the other shoals where dredging was done, the dredged material was placed in a heavy dike and in three check dams extending from the opposite bank. This dike was placed along the left edge of cut and is at no place over 50 feet from the left bank. It is joined at intervals of about 500 feet with the bank by light cross dams. The three check dams, which extend from the opposite bank, are built heavy and contract the channel to 200 feet, further contraction being found unnecessary. The three dams are spaced about 300 feet apart, the one furthest down stream being opposite the foot of cut. The channel obtained at these shoals is probably the best on the river below Charleston; however, no such difficulties were encountered here as at some other shoals.

MATTHEWS SHOALS

Ten miles below Charleston. Length obstructed, 3,350 feet; total fall, 2.0 feet; mean fall, 3.1 feet per mile; maximum fall previous to improvement, 18 feet per mile; maximum fall after improvement, 5.3 feet per mile; maximum current velocity previous to improvement, not known; maximum current velocity after improvement, 4.25 feet per second.

The maximum current velocity at these shoals is the maximum below Charleston. These shoals have always proved to be a serious obstruction to low water navigation, probably a more serious obstruction than Black Bird Shoals, certainly less susceptible to improvement. Several methods of improvement have been employed, but all have given way to the system of longitudinal and cross dams. This system, which confines the channel to the left bank, is 1,200 feet long and contracts the channel to 150 feet. This system of dams has beyond a doubt improved the shoals, as the figures above will show, but the proposed depths have not been obtained. In the past, the greatest difficulties encountered here by boats ascending the river was the excessive current velocities. These excessive velocities were caused by an unwarranted contraction of the channel in an attempt to secure certain depths. These great velocities have been greatly diminished, but insufficient depths are still found at the head of the shoals. Operations to obtain greater

depths were begun late in 1910, but very little was accomplished due to the work being stopped by high water. It is now proposed

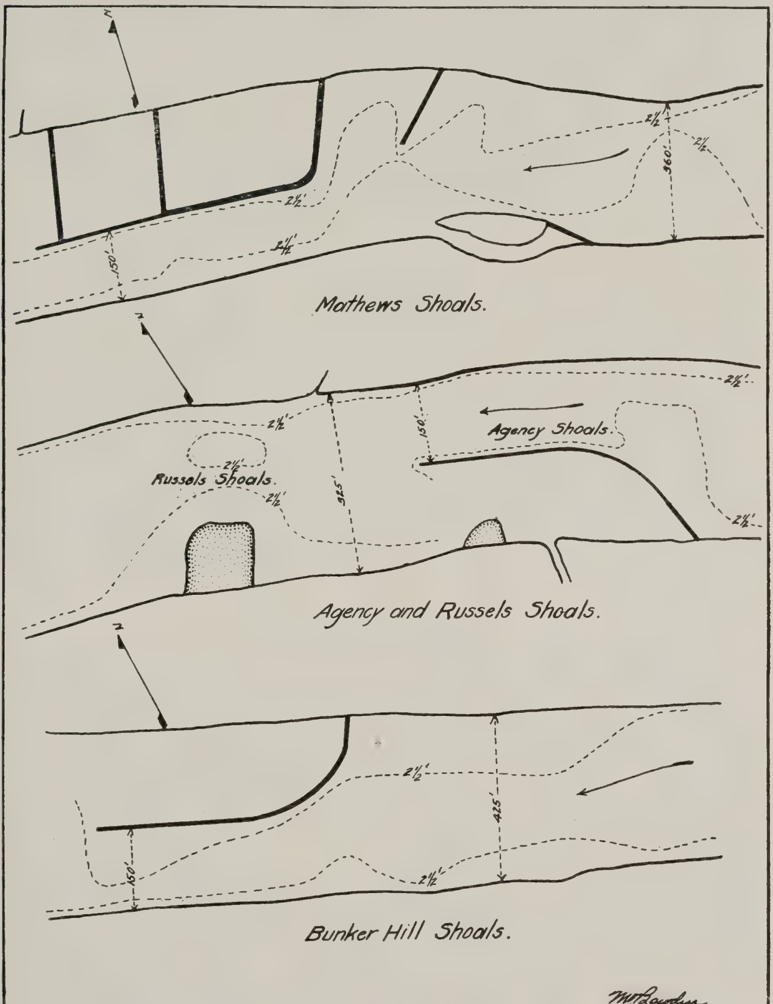


Plate III.

to excavate the rock and gravel at the head and thereby secure the proposed 30 inches.

AGENCY SHOALS

Eleven miles below Charleston. Length obstructed, 1,350 feet;

total fall, 0.8 foot; mean fall, 3.1 feet per mile; maximum fall previous to improvement, 12.1 feet per mile; maximum fall after improvement, 7.3 feet per mile; maximum current velocity previous to improvement, not known; maximum current velocity after improvement, 3.41 feet per second.

At these shoals the regulated channel has been confined to the right bank and contracted to 150 feet by a longitudinal dam connected to the left bank by a long wing dam. The total length of these dams is 900 feet. The result here has been satisfactory.

RUSSEL SHOALS

Eleven and one-half miles below Charleston. These shoals are only a short distance below Agency Shoals and are sometimes considered as being a part of them. The only obstruction here was a shallow gravel bar extending across the river. A channel 90 feet wide and 400 feet long was dredged along the right bank, and the excavated material was dumped on the opposite side of the river. The fall here is inappreciable and the current velocity is small. The improvement here is precluded from being an entire success because the dredged cut was not extended a sufficient distance down the river.

BUNKER HILL SHOALS

Fifteen miles below Charleston. Length obstructed, 1,650 feet; total fall, 1.2 feet; mean fall, 3.8 feet per mile; maximum fall previous to improvement, 13.2 feet per mile; maximum fall after improvement, 9.0 feet per mile; maximum current velocity previous to improvement, not known; maximum current velocity after improvement, 3.69 feet per second.

The improvement of the shoals has been effected in the same manner as at Agency Shoals by building a longitudinal dam connected to the bank by a long wing dam. The channel is contracted to 150 feet and confined to the left bank. As at Agency Shoals, the longitudinal dam is used without the check or cross dams. It is thought that better results would have been obtained at both shoals mentioned had the cross dams been built. The length of the dams at Bunker Hill Shoals is 900 feet. The improvement here has been satisfactory, but it is thought that the longitudinal dam should have been extended a few hundred feet farther down stream in order to hold more water where the foot of the dam is

now. The depths at the foot are about $2\frac{1}{2}$ feet, and any instability in the channel above might cause a decrease in these depths.

In order that the above descriptions may be more easily understood they are supplemented with the sketches shown in the Plates I, II, and III. These sketches show the bank lines, locations of dikes and dams, and bottom contours, but are rough, being drawn to no correct scale.

This 19-mile stretch of the river below Charleston, Tenn., in which the above ten shoals lie, has been improved, under the present project, by a total expenditure of \$78,966.15. This amount divided over the 19 miles gives a unit cost of improvement of \$4,156.11 per mile.

Guard Locks in Canals Connecting Tidal Bodies of Water

BY

Maj. EARL I. BROWN
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The question has frequently arisen as to whether a guard lock should be built in a canal connecting bodies of water subject to tidal influences. The arguments in favor of such locks being that the differences of levels caused by the tidal variations would cause excessive current velocities in the canal, with resulting hindrance to navigation and erosion of the canal banks.

A canal of quite large cross section having been constructed under my direction between Pamlico Sound and Beaufort Inlet, N. C., without guard locks, some observations were made by me in June, 1911, for the purpose of determining the tidal conditions therein, and the resultant tidal velocities.

The observations were begun upon receipt of a communication from Col. W. M. Black, Corps of Engineers, requesting such data about tidal conditions in this canal as were available for use of a board in considering a similar problem. Time and facilities were lacking to make observations as full and complete as desired, but the data obtained is submitted for such value as they may have.

The following is a description of the waterway: Beginning at Beaufort Harbor, it consists of a channel dredged to 250 feet in width and 10 feet in depth, lying in Newport River, and extending up same to the mouth of Core Creek. Newport River is a large tidal basin into which numerous small tidal creeks empty. It is split by shoals and reefs into several independent channels. The waterway follows the largest one of these channels. The lower end of Core Creek was originally wide and shallow, with extensive oyster shell reefs and mud banks. The channel dredged through this section is 125 feet wide and 10 feet deep. About 2 miles from its mouth the creek narrowed up to about 20 to 30 feet, and from that point the canal was dredged 90 feet wide and 10 feet deep

through the intervening farms and swamps to the head of Adams Creek. The latter was a fresh water stream rapidly widening out to about 1 to 2 miles, and about 8 feet deep. It is a tributary of Neuse River.

Beaufort Harbor, Newport River, and Core Creek are all tidal

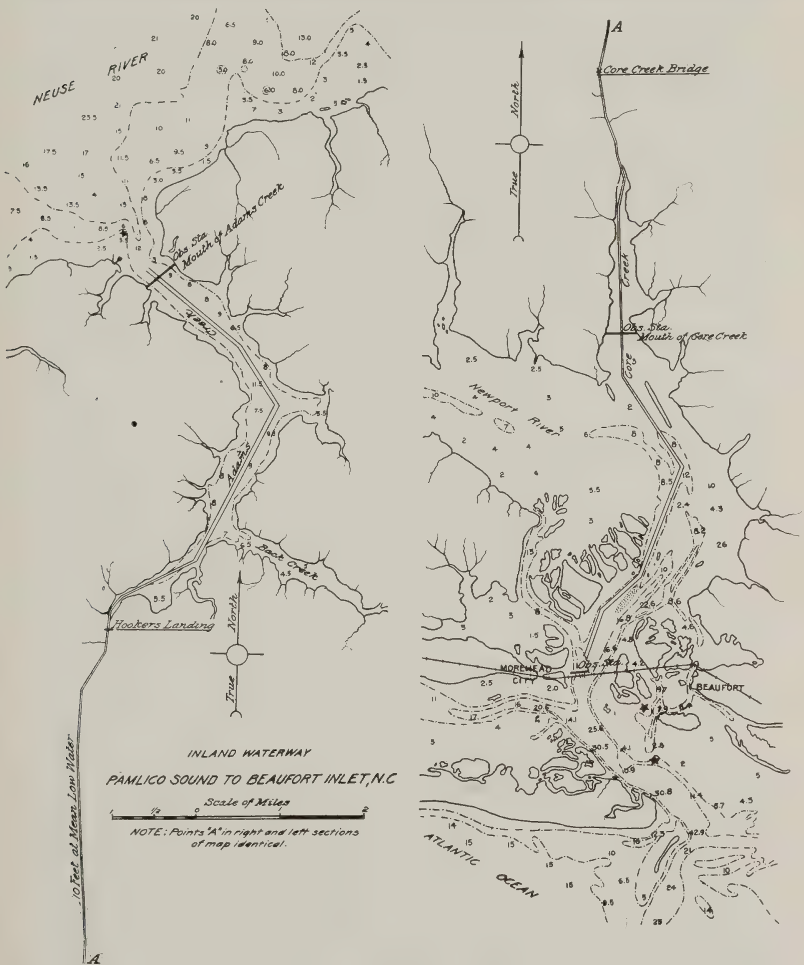

Plate I.

bodies. Neuse River and Adams Creek are not, their surface elevations being controlled only by the winds. Mean low water at the head of Adams Creek was ascertained to be 1.4 feet above mean low water in Beaufort Harbor by observations made some time ago.

Since the construction of the canal the situation is as follows: the tidal oscillation in Beaufort Harbor causes the level of the water at the mouth of Core Creek to rise above, and to fall below the level at the head of Adams Creek, so that ordinarily there is no permanent excess of elevation at either end of the canal.

The average tidal oscillation in Beaufort Harbor is 3.8 feet, and the average variation due to winds in Adams Creek is about 6 inches on either side of the mean level, with extreme variations of probably 3 feet above and 2 feet below the mean level.

A reference to the map accompanying this article will make this description clear.

Five stations were selected on the waterway, approximately 4 miles apart, at Morehead City, at the mouth of Core Creek, at the bridge over Core Creek, at Hookers Landing, at the head of Adams Creek, and at the mouth of Adams Creek. Gages were established at these points, all set to the same datum, mean low water in Beaufort Harbor, and continuous readings were taken for five days at fifteen minute intervals. From these readings the instantaneous tidal curves were plotted for the desired times.

As there was but one current meter available, simultaneous current measurements could not be taken at all stations, but a set of observations were made at each station, with meter readings every thirty minutes, taken throughout one entire tide, the readings being taken at Morehead City, June 18; at the mouth of Core Creek, June 19; at Core Creek bridge, June 20; at Hookers Landing, June 21, and at the mouth of Adams Creek, June 22, 1911.

The meter was provided with an electrical registering device, and had been carefully rated before the observations were begun.

In taking the observations, a small boat was provided with the necessary outriggers and anchored securely in the middle of the channel in such manner as to prevent it from moving out of position. The meter was lowered to six-tenths the depth at the section to obtain the mean velocity, which is meant in each case. It was allowed to run continuously, the registering device being put into commission, and readings taken every thirty minutes.

The results of these observations are plotted on the sheets giving the instantaneous tidal curves for the corresponding date. An effort was made to begin a set of observations at one low water and continue until the next.

The instantaneous tidal curves as given in the illustrations show some very peculiar characteristics: That Hookers Land-

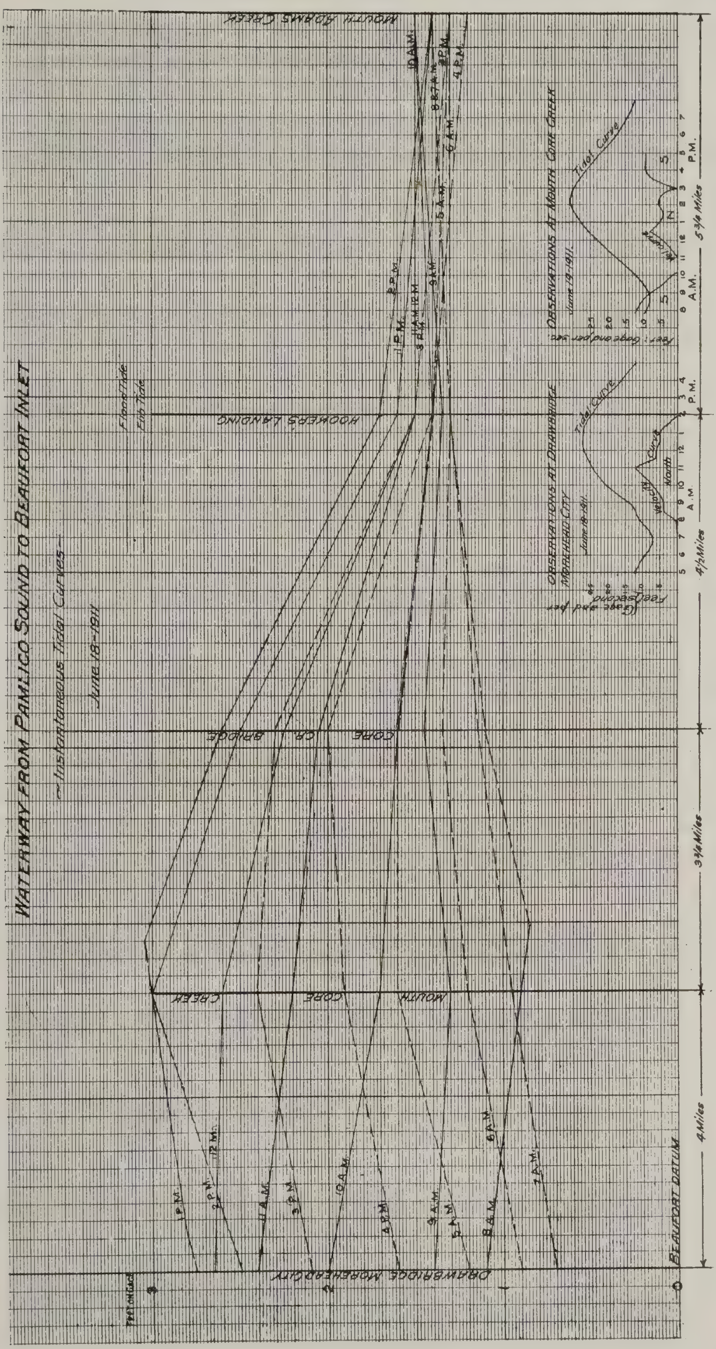
WATERWAY FROM PAMLICO SOUND TO BEAUFORT INLET
—Instantaneous Tidal Curves—
June 18-1911
Flood Tide
Ebb Tide
MOUTH ADAMS CREEK
HOOKERS LANDING
CORE CR. BRIDGE
MOUTH CORE CREEK
DRAWBRIDGE MOREHEAD CITY
BEAUFORT DATUM
OBSERVATIONS AT MOUTH CORE CREEK
June 19-1911
OBSERVATIONS AT DRAWBRIDGE MOREHEAD CITY
June 18-1911
Tidal Curve
4 Miles
3 3/4 Miles
4 1/2 Miles
5 3/4 Miles

Fig. 1.

ing is about the limit of any increase of elevation from tidal propagation which can be differentiated from surface variations caused by winds in Adams Creek; that the reservoir action of Adams Creek is sufficiently great to absorb the entire tidal prism propagated past Hookers Landing without appreciable increase in elevation; that the inflow into Adams Creek through the canal at high tidal stages is probably greater than the outflow at low stages; that high and low water occur about one hour later at the mouth of Core Creek than at Morehead City draw, but in the canal high and low water occur almost simultaneously throughout its entire length between the mouth of Core Creek and Hookers Landing; that the preponderance of tidal flow is north; that the greatest slopes and the highest velocities are found between the bridge and Hookers Landing; that the surface slope from Hookers Landing to the mouth of Core Creek is nearly always uniform no matter what the stage of the tide is, the slope varying with the stage of the tide, and with the wind variations in Adams Creek.

The section between the bridge and Hookers Landing having the greatest slopes and velocities is the one considered in the following remarks. It comprises that part of the canal connecting the two original bodies of water.

The cross sectional area at Hookers Landing, which is an average section, is 1,961 square feet, the wetted perimeter is 186.5 feet, making the mean hydraulic radius 10.5 feet. The slope at the time of maximum velocity on flood tide on June 20, 1911, as given for 4 p. m. on instantaneous profile is $.9' \div 5{,}280 \times 4\frac{1}{2} = 1 / 26{,}400$. Substituting this in the formula $V = C\sqrt{rs}$ we find $C = 125.36$. Using this value of C we find the resulting velocities for various differences of gage readings at the bridge and Hookers Landing as follows: 1 foot difference, 2.5 feet; 2 feet difference, 3.7 feet; 3 feet difference, 4.6 feet; 4 feet difference, 5.3 feet per second.

The following are maximum and minimum gage readings ever recorded at the localities named:

	Extreme low water	Extreme high water	M. L. W.
Mouth of Core Creek	—0.133	+3.367	+0.150
Core Creek bridge	—0.228	+3.660	+0.267
Hookers Landing	—0.420	+3.180	+1.357
Mouth Adams Creek	—0.288	+2.612	+1.465

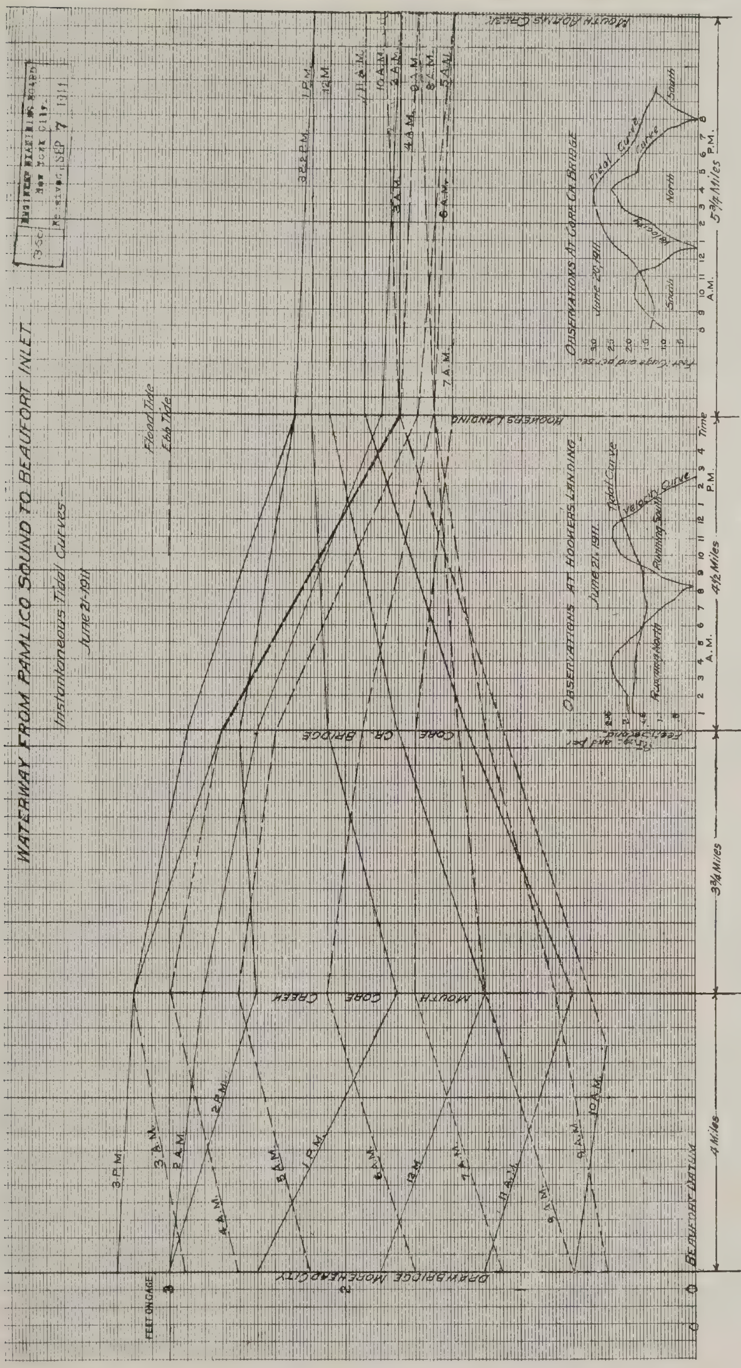
WATERWAY FROM PAMLICO SOUND TO BEAUFORT INLET.
Instantaneous Tidal Curves
June 21-1911
Flood Tide
Ebb Tide
Mouth Adams Creek
Hookers Landing
Core Cr. Bridge
Mouth Core Creek
Drawbridge Morehead City
Beaufort Datum
Feet on Gage
Observations at Core Cr. Bridge
June 20, 1911
Observations at Hookers Landing
June 21, 1911
Tidal Curve
Velocity Curve
Running North
Running South
5¾ Miles
4½ Miles
3¾ Miles
4 Miles

Fig. 2.

These elevations all refer to the established Beaufort datum, and the period of observations extended over about three months. The observations were made, however, before the canal was excavated, and it is not considered probable that the extreme high stages will ever be reached again, as they were caused by flood or storm conditions which caused a heaping up of the waters. Since the construction of the canal, this heaping up can not occur.

It will be noted that an extreme high water at Core Creek bridge occurring simultaneously with an extreme low water at Hookers Landing, if possible to occur, would result in a difference of elevation of 4.08 feet, and cause velocities of 5.5 feet per second, which would probably cause violent erosion of the banks and would be detrimental to navigation. Ordinarily, however, this difference does not exceed 1 foot and the usual maximum velocity is 2.5 feet per second.

If the canal were of greater length, the slope would be less, and the resulting velocity would be less; and on the other hand, if the canal were shorter the slope and velocity would be correspondingly greater, but the rate of change of the velocity is smaller than that of the slope, as it varies with the square root of the slope.

If the extreme conditions as given above, causing a velocity of 5.5 feet per second, or 3¾ miles per hour, were of common occurrence the banks would not stand, and a guard lock would be necessary. Now, if Neuse River and Adams Creek were tidal streams, conditions equal to these extremes might exist, provided original conditions had been such that high tide at the head of Core Creek corresponded with low tide at the head of Adams Creek, and a guard lock would be a necessity. On the other hand, if the times of high and low water at the two points named and under the assumed conditions should correspond, the velocities would be moderate in the connecting canal, and no guard lock would be necessary.

The canal has now been completed about nine months and this velocity, ordinarily about 2.5 feet per second, under normal conditions such as prevailed at the time of the observations has been sufficient to keep the channel scoured out quite well, and has caused but little erosion of the banks. Between the bridge and Hookers Landing the banks are about 10 feet in height above mean low water, the top 2 feet being rich swamp muck, and the remainder sand and shells. At and below the bridge the banks are low and marshy, with a heavy growth of marsh grass growing on them. Velocities any higher than those now existing might become trouble-

some, but those now prevailing have not yet caused any damage. These velocities seem to have but little effect on boats navigating the canal, except that sailboats sometimes await a favorable set of the tide before starting out.

An examination of the complete records as taken during these observations show that on each tide the average time the gage reading was greater at the bridge than at Hookers Landing was $8\frac{1}{2}$ hours, and the average time they were greater at Hookers Landing than at the bridge was $3\frac{3}{4}$ hours. Assuming that the change of direction of flow of the current follows the change of slope at about the same interval in each case, the time that the current sets to the northward is therefore more than twice as long as that of its set to the southward. This resultant flow northward has caused Adams Creek to become brackish throughout its entire extent where it was formerly fresh.

Summing up, it appears that no conditions are likely to arise on this canal to require guard locks to be built, on account of Neuse River and Adams Creek being non-tidal.

Guard locks will probably be necessary in short canals connecting tidal bodies where there is much difference of time in high and low tide at the extremities of the canal, but if the canal were sufficiently long, the slope might be decreased enough to reduce the velocity to a desirable rate, and the guard lock avoided. This length will, of course, depend upon the amount of tidal variation at the two ends.

Water Supply of the District of Columbia

BY

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Corps of Engineers

SOURCE AND DISTRIBUTION.

The source of supply is the Potomac River about three quarters of a mile above Great Falls, approximately 14 miles above the city. Below the headworks of the aqueduct the river bed drops rapidly to tidewater at the foot of Little Falls, 9½ miles distant, the fall being approximately 150 feet. Both the falls mentioned would be more accurately called rapids, at which occurs most of the 150-foot fall in the river bed. At the headworks of the aqueduct a low masonry diversion dam extends across the river from the Maryland to the Virginia shore. This dam fixes the elevation of the water at the head of the system, which elevation varies ordinarily during the year as much as 3 feet, depending on the stream discharge. The water enters the intake at the Maryland end of the dam, passes through a short feeder under the Chesapeake & Ohio Canal to a gate chamber, thence to the conduit, which has a ruling diameter of 9 feet and a slope of 0.792 foot to the mile. Passing through the first section of the conduit, 9 miles in length, the water is discharged into the first reservoir, now called the Dalecarlia Reservoir, which is approximately ½ mile long between its inlet and outlet. From the outlet of this reservoir the water passes through another section of conduit, 2 miles in length, to the Georgetown Reservoir, formerly known as the distributing reservoir. The distance between the inlet and outlet of this reservoir is nearly ½ mile. From the outlet the water enters the west vertical shaft of the Washington City tunnel, sometimes called the "Lydecker Tunnel," the lowest point of which is 29 feet below sea-level. Passing through this tunnel, 4 miles in length, the water emerges from the east vertical shaft into the receiving chamber of a gatehouse over the shaft and discharges thence through a "circulating" conduit 9 feet in diam-

eter to the upper end of the McMillan Park Reservoir, from the lower end of which the water is lifted by pumping to the filters.

The distance traveled to the filters by the water is 16½ miles. The difference of elevation (or loss of head) between the water surface at the head of the system and the water surface in the McMillan Park Reservoir depends, of course, on the rate of removal from the reservoir, but for an average rate of sixty million gallons daily would be 6.4 feet.

Passing through the filters the water is collected into a small (fifteen million gallons) covered filtered-water reservoir, and thence passes through four 48-inch cast-iron mains laid on the bottom of the McMillan Park Reservoir to the distributing chamber in the East Shaft Gate House.

From the latter place the water enters the distribution system, and up to and including the time and place of entrance into the distribution system the water, and all the works connected with handling the water, are, by Act of Congress, placed under the control and direction of the Chief of Engineers, U. S. Army, who is represented by an officer of the Corps of Engineers, U. S. Army, in immediate charge. After entrance into the distribution system the water, until it reaches the consumer, is under the control of the Water Department of the District of Columbia. This Department is under the administration of the Engineer Commissioner, one of the three commissioners forming the administrative body of the District of Columbia. The Engineer Commissioner is by law required to be an officer of the Corps of Engineers, U. S. Army, but is entirely removed from any control by the Chief of Engineers while acting as Engineer Commissioner.

From the gatehouse mentioned above, which is the head of the distribution system, the water enters four cast-iron mains, through one of which (a 75-inch main) water is supplied to the gravity service of the distribution system, and through the other three (48-inch mains) water is supplied to the high service pumping station of the District, located west of and adjacent to the south end of McMillan Park, the site of the last reservoir in the aqueduct system and of the filtration plant. The District of Columbia, in addition to the area supplied by the gravity service, is divided into four high-service areas, supplied from the District pumping station through the three 48-inch mains mentioned above.

The distribution of the water after purification is under the District Water Department, and all expenses of operation, main-

tenance, and extension of the distribution system are paid for, as required by an Act of Congress, from the revenues of the Water Department, except such items of extension as are otherwise provided for from time to time in the annual acts of Congress providing for the expenses of the government of the District of Columbia.

The construction of the system from Great Falls to and including the Georgetown Reservoir was paid for entirely by the Federal Government. The remainder of the aqueduct system and also the cost of the operation and maintenance of the entire system since 1878 has been paid equally from the revenues of the District of Columbia and of the United States. In addition, the original distribution mains from the Georgetown Reservoir to the Federal buildings in the city were paid for by the United States, but the District of Columbia authorities were given permission, under restrictions, to tap the supply mains for the use of the inhabitants of the District of Columbia, with the provision that no greater number of mains should be laid at the expense of the United States than are sufficient to supply the needs of the public buildings and grounds. An Act of Congress also provides that when the supply of water is no more than adequate for the needs of the General Government, the supply to the inhabitants of the District shall be stopped. Under this latter provision, which has not been repealed, the inhabitants of the District of Columbia, and therefore the Commissioners (the administrators of the District) are more deeply concerned in the possibility of a shortage of supply than the administrators of the Federal bureaus, because the enforcement of the law will require the curtailment of the supply to the inhabitants of the District and an undiminished supply to the Federal buildings, when the demands shall exceed the supply.

HISTORY.

Prior to the construction of the Washington Aqueduct, the National Capital had no municipal water supply system, the residents drawing their supplies from springs or shallow wells. In 1852 Congress appropriated $5,000 to enable the President to cause an investigation to be made to determine the best means of affording the cities of Washington and Georgetown ''An unfailing and abundant'' supply of water.

Under the act mentioned above, Lieut. M. C. Meigs, Corps of Engineers, was ordered to carry out the wishes of Congress, and under date of February 12, 1853, he submitted his plans and esti-

Fig. 1. Cabin John Bridge in its completed state. View taken 1892. This is one of the longest masonry arches in the world, being 220 feet between springing lines. It was begun in 1858 and practically completed in 1861, though the aqueduct was not completed across it until 1863. Designed and built under Gen. (then Captain) M. C. Meigs, Corps of Engineers.

mates to Congress, through military channels, in which he recommended the construction of an aqueduct from the Potomac River at Great Falls to obtain the kind of supply directed by Congress.

Under authority contained in the Act of Congress passed in 1853 making an appropriation to begin construction, the President approved the project of Lieutenant Meigs which provided for an aqueduct 9 feet in diameter from Great Falls, and Lieutenant Meigs was ordered to take immediate charge of construction.

Since the beginning of the work, the construction, operation, and maintenance of the aqueduct has been under the direction of an officer of the Corps of Engineers of the Army, except for a period of five years during and just immediately following the Civil War when all officers were needed for active duty with the Army. During this interim the aqueduct was placed by Act of Congress under the Interior Department.

Lieutenant Meig's original project contemplated a dam at Great Falls across the full width of the river to hold the water at a proper level, whence it was to be conducted by a circular conduit of brick 9 feet in diameter having a fall of 0.792 foot (a little more than 9½ inches) to the mile, discharging, about 10 miles below Great Falls, into a receiving reservoir which was to act as a storage reservoir and a sedimentation basin. From there the water was to be led through another section of the conduit, about 2 miles long, to the distributing reservoir around which a by-conduit was to be constructed to be used during times of repairing, or in case of accident to the reservoir, requiring its temporary disuse. From the distributing reservoir mains of proper size were to be laid from time to time as necessary to distribute the water through the city.

Water was first admitted to the system in 1859 from the Little Falls Branch, discharging into the receiving reservoir, after which the section of the aqueduct above the receiving reservoir was constructed and completed to such an extent that it was possible to admit the Potomac water to the system in December, 1863. The diversion dam at the intake at Great Falls was built as a temporary dam of riprap and extended from the Maryland shore to an island in the river, thus permitting the river to discharge unobstructed through its channel between the island and the Virginia shore. The riprap dam was replaced by a masonry dam begun in 1864 and completed in 1867.

The original project contemplated the use of the water from Little Falls Branch, in order that early benefit should be derived

Fig. 2. View of Cabin John Bridge during erection in 1858 showing centering for arch, scaffolding, and derricks.

from the construction. It was intended, however, that the structures to Great Falls should be completed as rapidly as possible to obtain Potomac water, as the estimated discharge of Little Falls Branch was but two or three million gallons daily, whereas the estimated consumption for the population of 58,000 was five and a quarter million gallons daily for public and domestic purposes. Probably on account of the economies necessary in order to meet the large expenses of the Civil War, the appropriations were limited and progress towards completion consequently slow.

As the water of Little Falls Branch discharging into the receiving reservoir was believed to be contaminated because of the number of inhabitants living in its catch basin, a by-conduit was, about 1863, added to the original project to pass water from Great Falls around the reservoir. By this means it was intended that the storage of the reservoir should be used only in case of high turbidity of river water, or in case of accident above the reservoir. This by-conduit was constructed beween 1864 and 1867.

It seems that the water stored in the reservoir was seldom used after the construction of the by-conduit, probably because the turbidity created by the discharge from the Little Falls Branch in most cases coincided with the turbidity in the river, and also because there was a general belief that the stored water was impure and unfit for drinking.

As the consumption increased in the city and the suitability of the stored water for drinking purposes diminished with the increase of population in the catch basin above the reservoir, the necessity for the diversion of the small streams discharging into the reservoir and its restoration to continuous use in the system became more urgent. The construction of works necessary to accomplish this was recommended and estimates of cost were submitted by the officer in charge in 1891. Appropriations were made by Congress in 1893 and 1895, and the work was completed in 1896. The restoration of the reservoir slightly increased the capacity of the aqueduct in its lower section by eliminating a loss of head of about 1 foot occurring when the by-conduit was in use.

The necessity for the extension of the dam at Great Falls to the Virginia shore was recognized by the officers in charge, and estimates were submitted annually after 1867 until appropriation was made for the purpose in 1882. The completion of this dam in 1886 marked the completion of the essential features of the original project affecting the capacity of the aqueduct.

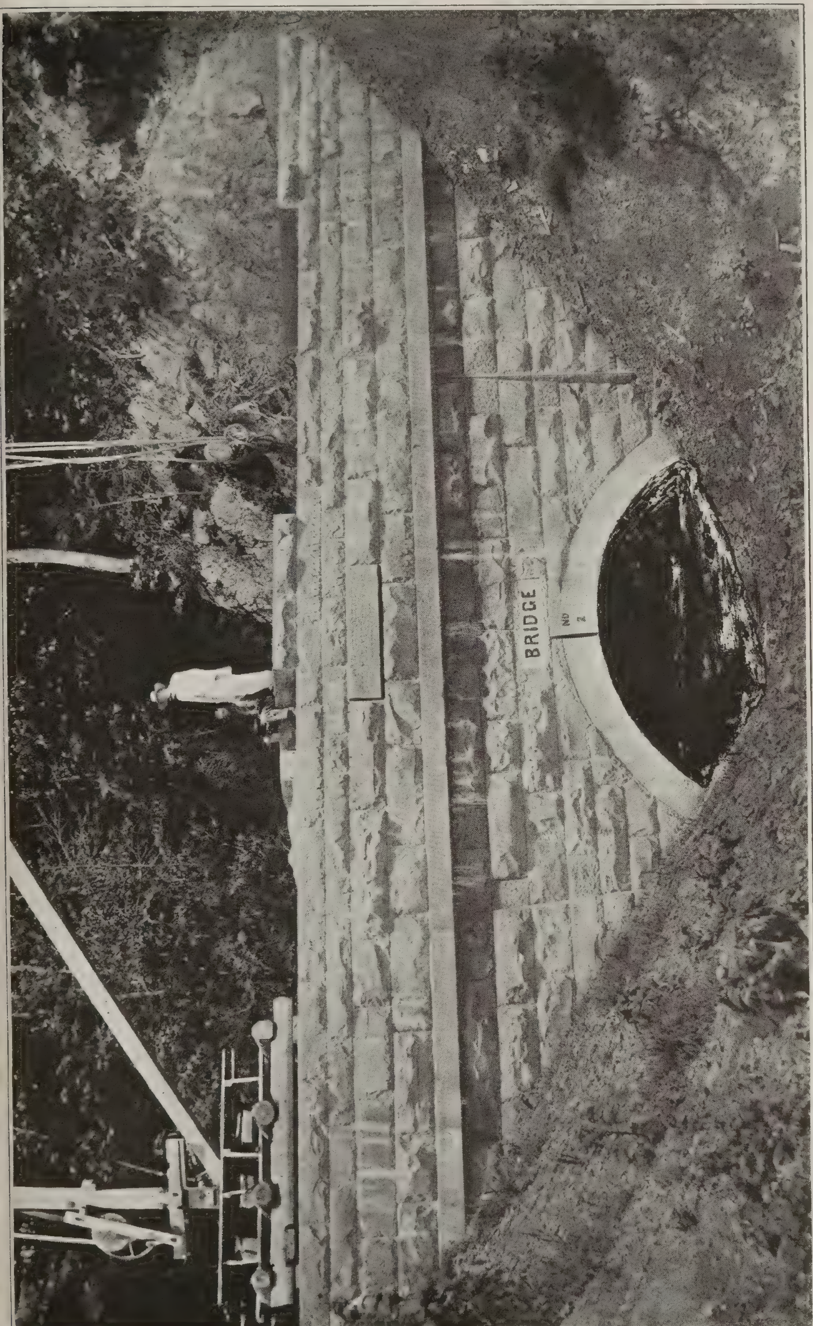


Fig. 3. One of the stone arch bridges carrying aqueduct over small streams, etc.

The project for the increase of the water supply (1882) which included the extension of the dam to the Virginia shore, also provided for the extension of the lower end of the aqueduct by a tunnel to a new reservoir, the site of which is now known as McMillan Park, in the vicinity of the Soldiers Home. The latter work was begun in 1882, suspended in 1888, and completed in 1902.

To meet the increased demands of the city for water, the capacity of the conduit was increased by raising the dam at Great Falls 2½ feet to its present height (150.5 feet above Washington Aqueduct datum), and by the removal of sedimentary deposits in the conduit. These items were recommended in 1891 and 1892—appropriation made by Congress in 1895—and work completed in 1896.

At the distributing end of the system the Filtration Plant was constructed and put into operation in 1905.

The only works materially affecting the capacity of the conduit since its completion in 1863 have been the replacement of the temporary dam across the Maryland channel of the river by a permanent dam in 1867, the extension and raising of the dam in 1886, and the further increase in height of the dam in 1896. The restoration of the receiving (Dalecarlia) reservoir to the system slightly increased the capacity of the aqueduct, as indicated in a previous paragraph.

However, the full capacity of the aqueduct to deliver water to its lower end could not be utilized until the extension of the system to McMillan Park and the establishment of the Filtration Plant, on account of the necessity for maintaining the level in the distributing reservoir at Georgetown at 144.0 to maintain pressures in the city's distributing system. This limiting level gave a maximum capacity of 76,500,000 gallons daily, after raising the dam at Great Falls in 1896, whereas now, the elevation of the water at the head of the distributing system, after the water passes the filtration plant, being constant, the pressures on the distributing system are independent of the levels of water in the reservoir, and, therefore, the full capacity of 90,000,000 gallons daily can now be considered available for short periods.

The storage capacity of the system was increased by the restoration of the Dalecarlia Reservoir and the addition of the McMillan Park Reservoir, thus giving a reserve supply to meet demands in excess of the capacity for short periods. This advantage was apparent in the winter of 1904-1905, when the waste of water became so excessive that had it not been for the reserve supply of the

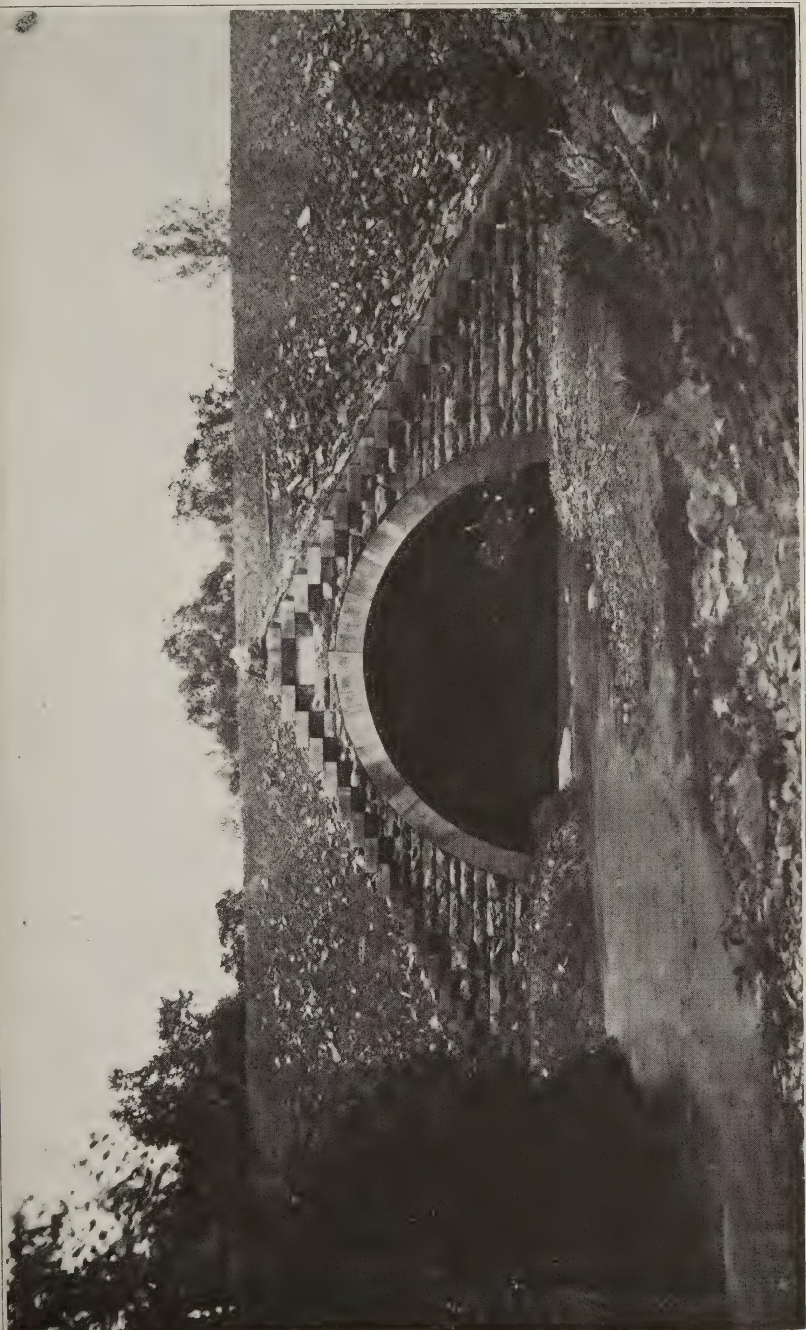


Fig. 4. Another style of arch bridge along line of aqueduct.

reservoirs there would have been a serious shortage of water in the city for several days.

Between 1863 and 1867, in the operation of the aqueduct difficulty was experienced in keeping the water at a proper level at the intake to maintain the water in the distributing reservoir at an elevation sufficient to maintain pressures on the distribution system, on account of leakage through the riprap dam, and, at various times between 1868 and 1886, on account of the lack of a permanent dam to raise the level of water in the Virginia channel, the tendency being for most of the flow of the river to be discharged through the latter channel during periods of low water. Nevertheless, the flow in the conduit was maintained in sufficient quantity to furnish water at the lower end of the system as fast as the distributing mains could take it away, first, by maintaining the temporary riprap dam in the Maryland channel and after its replacement in 1867 as the consumption increased, by the construction and continual repair of a temporary dam built into the Virginia channel from the upper end of the dividing island (Conns) to divert the water into the Maryland channel of the river.

Never since 1863 has there been an interruption of the supply of water to the distribution system of the city. To decide whether or not the supply has been abundant, one need only examine the records of per capita consumption, which in the early years of the aqueduct appears to have been about 100 gallons per day, and which has gradually increased until the maximum of 218 was reached in 1905, which is among the highest recorded for cities in the United States. That the supply of water has been ''unfailing and abundant'' for fifty-one years, during which the population has increased from 60,000 to 343,000, indicates how well the designer and those in charge since construction have complied with the instructions of Congress.

In fact, the supply has been so abundant that the system has always been able to furnish water in quantities sufficient to meet the abnormal and excessive waste which has existed for many years. The useless waste of water in the city was recognized as early as 1868, when the officer in charge recommended the installation of meters in the portion of the city supplied by the old Georgetown High Service Reservoir, to reduce the waste of water pumped to that reservoir. As the demands of the city have in the past made necessary the consideration of increasing the supply, the advantages of the use of meters to reduce the illegitimate consumption

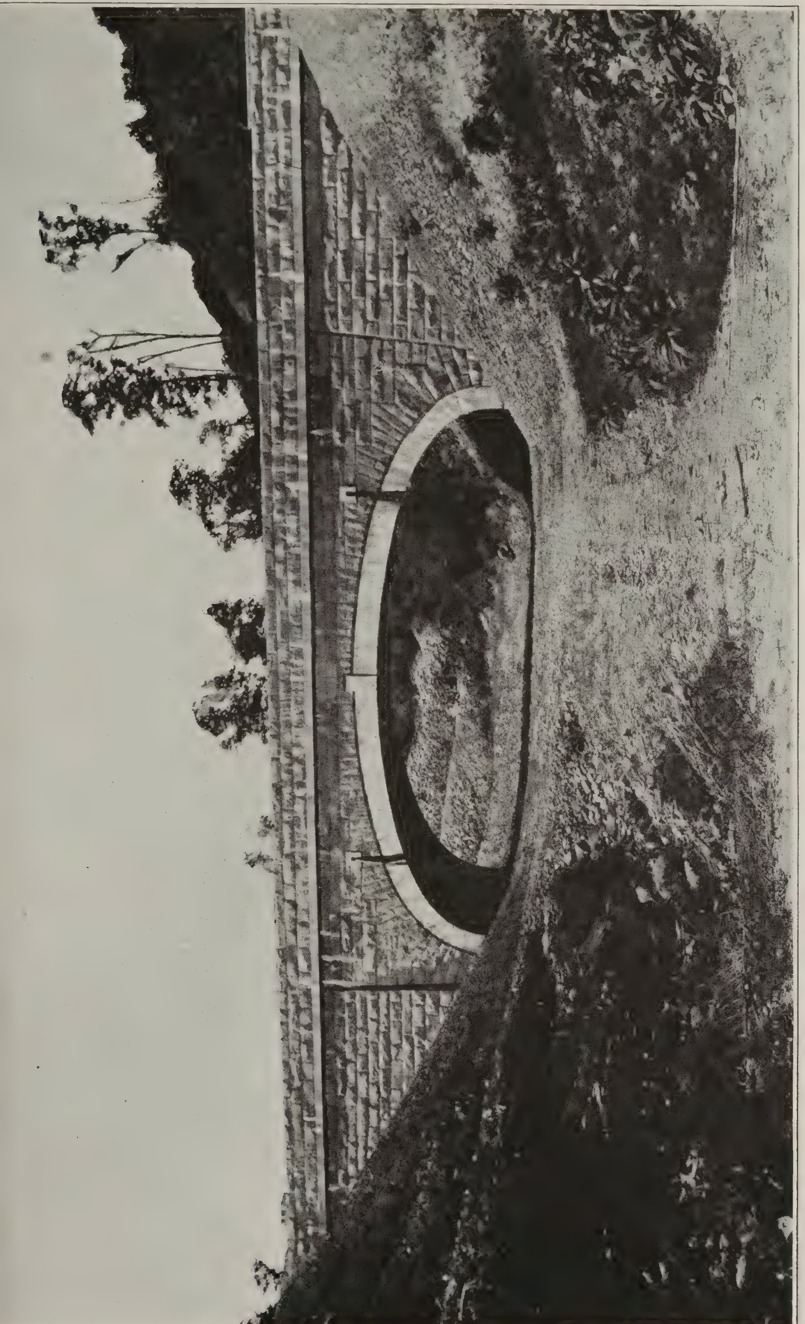


Fig. 5. A third style of arch bridge along line of aqueduct.

and waste of water has always been recognized and reported by the officer in charge. Opposition to the use of the meter prevented its general introduction, resulting in steps being taken on every occasion to meet the waste by increasing the capacity of the aqueduct system or by increasing the capacity of the distributing system. Universal metering to check the waste of water has frequently been recommended in the annual reports of both the officer in charge of the Washington Aqueduct, and the officer in charge of the District of Columbia Engineer Department, but without resulting in action by Congress to that end.

PRESENT CONDITIONS.

Finally, in February, 1905, the consumption of water for more than a week exceeded the capacity of the conduit, and it then became evident that radical steps were necessary to prevent a shortage of supply within a few years following. At the next session of Congress the opposition to meters was overcome by the urgency of establishing means of checking the waste until the supply could be increased, and $100,000 was appropriated to begin the installation of meters, and in 1907 authority was granted to expend surplus funds of the District of Columbia Water Department for extending the meter system. Other methods of locating leaks and preventing waste were also adopted by the District of Columbia Engineer Department.

In 1908 Congress directed that preliminary investigations and surveys to increase the water supply be made under the direction of the officer in charge of the Washington Aqueduct. The investigations covered not only the methods of increasing the supply to the city by constructing new works to conduct more water to the city, but also the possibility of avoiding such additional and expensive works by restricting the waste. The results of these investigations led the officer in charge to report that a new aqueduct would be necessary unless the waste of water should be greatly restricted, and as it appeared more economical to check the waste than to construct a new aqueduct, among other things recommended was the prompt installation of meters on all water services, the complete work to be provided for by three consecutive annual appropriations.

As the results accomplished by the Water Department were exceptional, not only the tendency to increased per capita consumption having been checked, but also the actual normal consumption

Fig. 6. Constructing the 9-foot brick aqueduct through a deep cut.

having been reduced in spite of the increase in the number of water services, the belief seemed to have become established in the District of Columbia Engineer Department that there was no need of haste in installing meters. This naturally caused a reduction of efforts to greatly extend the system of meters, so that for the fiscal year 1909 and 1910 the number of new services exceeded the number of new meters installed, the available funds of the water department having been used for purposes considered more urgent by the Water Department officials.

This postponement of the general installation of meters led the writer to make investigations to determine whether the conditions had been changed sufficiently to remove the possibility of the demand again exceeding the supply, and the dangers incident to such a condition. The inquiry has led to a study of the relation between air temperatures and the consumption of water, especially in the winter.

The consumption of water between January, 1903, and October, 1905, was determined by computing the daily rate of flow from the loss of head of water in the aqueduct between Dalecarlia and Georgetown reservoirs, and making the necessary corrections for the changes in the levels of the reservoirs. The data necessary for this purpose was obtained from the Washington Aqueduct records. The consumption has been metered and recorded at the Filtration Plant since 1905. From this data, together with data from the records of the Weather Bureau, the chart herewith has been prepared, showing the average daily consumption and average daily mean temperature by weeks as well as the average daily consumption by months, since January, 1903.

An examination of this chart shows two peaks in the curve of consumption of each year except 1909, the peaks occurring one in summer and the other in winter. An examination of the temperature curve shows an increase to a maximum in midsummer followed by a decrease till the curve drops below the heavy line representing freezing temperature (except for 1908-1909), sometimes moving back and forth several times across the line before the gradual spring rise sets in. It will be further noticed that the summer and winter peaks in the consumption curve are respectively coincident with the summer peak in the temperature curve and the fall of the temperature curve below the line of freezing, and that the failure of the peak to develop in the consumption curve in 1908-1909 is coincident with the failure of the tempera-

ture curve to fall below the freezing line, thus showing very strikingly the effect of air temperature on the consumption of water.

Examining the temperatures and consumption by weeks as plotted, it will be noted that the consumption decreases in the fall of the year until the occurrence of freezing temperatures. If freezing temperatures fail to develop until late in December or in January, the consumption continues to decrease until freezing temperatures do develop, which is positive proof that the quantity of water used for *necessary and legitimate* domestic and public purposes is least in seasons of low temperatures, and if it were not for the practice of running off the water to counteract the effect of cold weather the consumption would be least about the end of winter.

The following are general characteristics of the curves: A decrease in temperature below freezing is coincident with or immediately followed by an increase in consumption which is followed by decrease in consumption at the time of or just after a rise in temperature above freezing. The amount of these changes in consumption varies with the extent of departure of temperatures from the line of freezing.

An increase in consumption caused by decrease in temperature below freezing is followed by further increases in consumption in each of the following weeks until the temperature rises again above freezing, the amount of these increases depending on the degree of cold.

The amount of waste, the difference between the actual and normal consumption, accompanying a period of temperature below freezing is greater at the end than at the beginning of winter.

Therefore, in comparing the excessive consumption of water occurring in different years, not only the number of weeks of freezing temperature should be considered, but also their arrangement and the degree of cold.

Temperatures having such an effect on the amount of water taken by the city, it is evident that the annual average daily consumption of water would in many cases be materially affected by this condition. As the degrees of heat or cold and the arrangement of the different degrees of heat or cold are not throughout any year similar to any other year, the effect on the consumption of water in any year is not the same for any other year. In order to determine tendencies from year to year, to increase or decrease in the normal consumption of water, a figure should be selected which would be independent of causes materially affecting the rate at

which the water is taken; in other words, a period selected in each year where this increased consumption due to temperatures is eliminated as much as possible. The month of November has been selected as such a period, because then the temperatures are usually above 32 and below 60, so that there is no incentive to run water from a faucet either to prevent freezing in the pipes or to draw off warm water to get a cool drink, and the date of killing frost occurring before November 1, the use of water for plant life is usually almost eliminated. The month of the year when the weekly mean consumptions depart least from the monthly mean is generally November.

The rate of consumption of water in the month of November serves as a basis to determine the amount of increase in the rate of consumption due to cold weather, and if the temperatures should be alike for the same period in winters of different years it would be possible to determine whether the system is becoming more or less sensitive to cold weather by noting the amount of change in the rate of consumption for the same change in temperature. If it should appear that the water system is not less sensitive than it was in the past, the probable increase in consumption above the normal would not now be less than it has been in the past for similar conditions of temperature.

TABLE 1.—*Fiscal Year.**

	1903	1904	1905	1906	1907	1908	1909	1910	1911
	Millions of gallons								
Average daily consumption for November	54.7	58.6	62.1	66.1	65.5	61.5	60.1	55.3	57.9
For year	58.0	61.1	68.7	67.4	66.9	64.9	61.5	59.2	60.4

*The fiscal year 1911 extends from July, 1910, to June, 1911.

Extensive metering began October, 1906 (fiscal year 1907) and pitometer service was organized a few months earlier. The above shows clearly the effect of efforts of the Engineer Department of the District of Columbia to reduce the waste of water. The average annual increase in the normal daily consumption for November prior to the fiscal year 1907 was about four million gallons. After the establishment of methods to reduce waste the increase was eliminated and reduction in consumption was effected, until the fiscal year 1911.

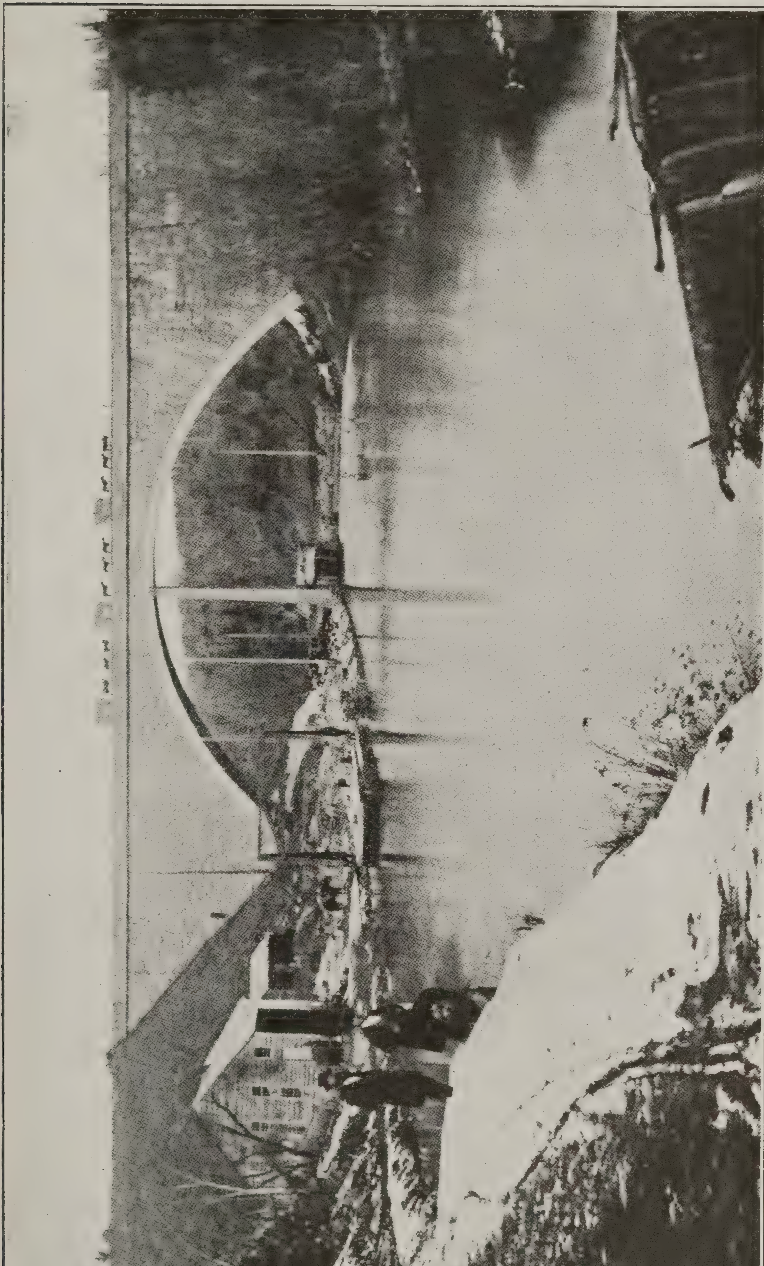


Fig. 7. Army supply wagons crossing Cabin John Bridge in 1861.

TABLE 2.—*Statement of Water Services in the District of Columbia. Corrected, 1909.*

June 30	Total number of services	Number of new services Fiscal year ending	Number of new meters Fiscal year ending	Total number of meters	Number of services unmetered	Percent of services metered	Amount of constant waste discovered and corrected by Pitometer Division. Gallons per day
1898	44,493	1,757	130	907	43,586	2.0	--------
1899	45,905	1,412	105	1,012	44,893	2.2	--------
1900	46,999	1,094	123	1,135	45,864	2.4	--------
1901	48,207	1,208	105	1,240	46,967	2.5	--------
1902	49,596	1,389	253	1,493	48,103	3.0	--------
1903	51,044	1,448	255	1,748	49,296	3.4	--------
1904	52,351	1,307	180	1,928	50,423	3.7	--------
1905	54,035	1,684	176	2,104	51,931	3.8	--------
1906	55,721	1,686	297	2,401	53,320	4.3	--------
1907	57,672	1,951	5,866	8,267	49,405	14.0	914,000
1908	59,532	1,860	4,344	12,611	46,921	21.0	5,604,400
1909	60,117	2,142	1,968	14,579	45,538	24.2	9,560,635
1910	63,472	2,687	1,361	15,940	47,532	25.1	6,364,190

An examination of the chart shows only one case in which there was a period in one year when the temperatures were the same and acted in a similar manner as in another year.

The following indicates the periods and resulting rates of consumption of water:

TABLE 3.

1906.	Temperature	Million gallons average daily consumpti'n	1907.
Last week in January	43	64	
	44	64	Second week in January
First week in February	34	66.6	
	34	67.5	Third week in January
Second week in February	27	72.2	
	27	71.9	Fourth week in January
Third week in February	33	68.7	
	31	73.7	First week in February

The temperatures prior to the above periods acted in an approximately similar manner in the two years. The system appears to have been no less sensitive in 1907 than in 1906, but as the extensive metering did not begin until October, 1906, it would probably not be proper to consider the results above to determine the effect of metering on the sensitiveness of the water system. There being

no case like the above in which one period occurred in a year prior to January, 1907, and the other period after June, 1908, between which dates about 10,000 meters had been placed, it is not possible by such direct comparison to determine the effect of the metering on the total waste in cold weather.

Arranging the winters in order of their severity and setting opposite the corresponding maximum increase in rate of consumption over the preceding November rate, we have the following table:

TABLE 4.

Winters	*Maximum increase.*
1. 1904-1905	28 million gallons.
2. 1903-1904	11 million gallons.
3. 1906-1907	10.5 million gallons.
4. 1909-1910	10.0 million gallons.
5. 1907-1908	9.8 million gallons.
6. 1905-1906	5. million gallons.
7. 1908-1909	0.0 million gallons.
8. 1910-1911 (incomplete)	9.0 million gallons to 3d week in December.
1910-1911 (addenda)	9.8 million gallons.

From this relation it appears reasonable to assert that the sensitiveness of the system is no less now than it has been in the past four years and is probably greater than it was in 1903-1904 because, although the winter of 1903-1904 appears much colder than 1909-1910, the increases due to cold weather differ very slightly.

The winter of 1904-1905 was unusually cold, not comparable with any other winter recorded on the chart. Granting that there is likely to be wasted in cold weather more water now than in 1903-1904 and not less than in 1906-1907, it is also probable that the water that would probably be wasted in case of a recurrence of a winter similar to that of 1904-1905 would not be less than in the latter winter. Therefore, in case of a recurrence of such a winter, the consumption in November, 1910, having been at the rate of fifty-eight million gallons daily, the maximum rate of consumption would be 58 plus 28, or eighty-six million gallons daily, which is very close to the maximum capacity of the conduit.

If the measures for the prevention of waste had not been effective since 1906, thus permitting the annual increase in consumption prior to 1906 of about four million gallons daily to continue, the demand for water would have exceeded the capacity of the conduit in the summer of 1909 and the following winter and summer. Assuming the deductions in the preceding paragraph to be correct, in spite of the waste prevented, the same condition would have

occurred during any winter since 1905, that of 1909-1910 probably excepted, had there been a recurrence of the winter of 1904-1905.

As the cause of the waste of water in cold weather is due to the passing of water through fixtures to prevent freezing, the amount of waste would probably be directly proportional to the number of water services, other conditions such as pressures, meters, etc., remaining the same. I believe this can be indicated by a comparison of the waste in February, 1899, and February, 1905, in each of which there was a period of three weeks in which the weekly temperatures acted in a similar manner, although the temperatures were lower in 1899, as indicated in Table 5.

TABLE 5.

1899.	Temperature.	Increase in rate of consumpti'n	1905.
January:			January:
Fourth week	37	------	
	33	------	Third week
February:			
First week	26	2.3	
	25	2.0	Fourth week.
Second week	14	4.0	
			February:
	17	10.0	First week
Third week	21	4.6	
	25	2.8	Second week
Fourth week	33	—8.5	
	22	+3.2	Third week

The total increase in 1899 for four weeks was 10.9 million gallons daily, and in 1905 was 14.8 million gallons daily. From Table 2 it will be seen that there were about 9,000 more services in 1905 than in 1899, the increase being about one-fifth. Therefore, if the average service in 1905 had wasted as much as in 1899 the total increase in 1905 would have been approximately 10.9 plus one-fifth of 10.9, or 13.1 million gallons daily, which is 1.7 million gallons daily less than the actual excess of 14.8 in 1905. As the number of meters was nearly the same in the two years, this difference can be accounted for by the increase of pressures on the distributing system between 1899 and 1905 which would cause an increase in waste from the average service.

That there is not in Table 4 positive and plain indication of an increase in waste in the last few years over the waste in a winter

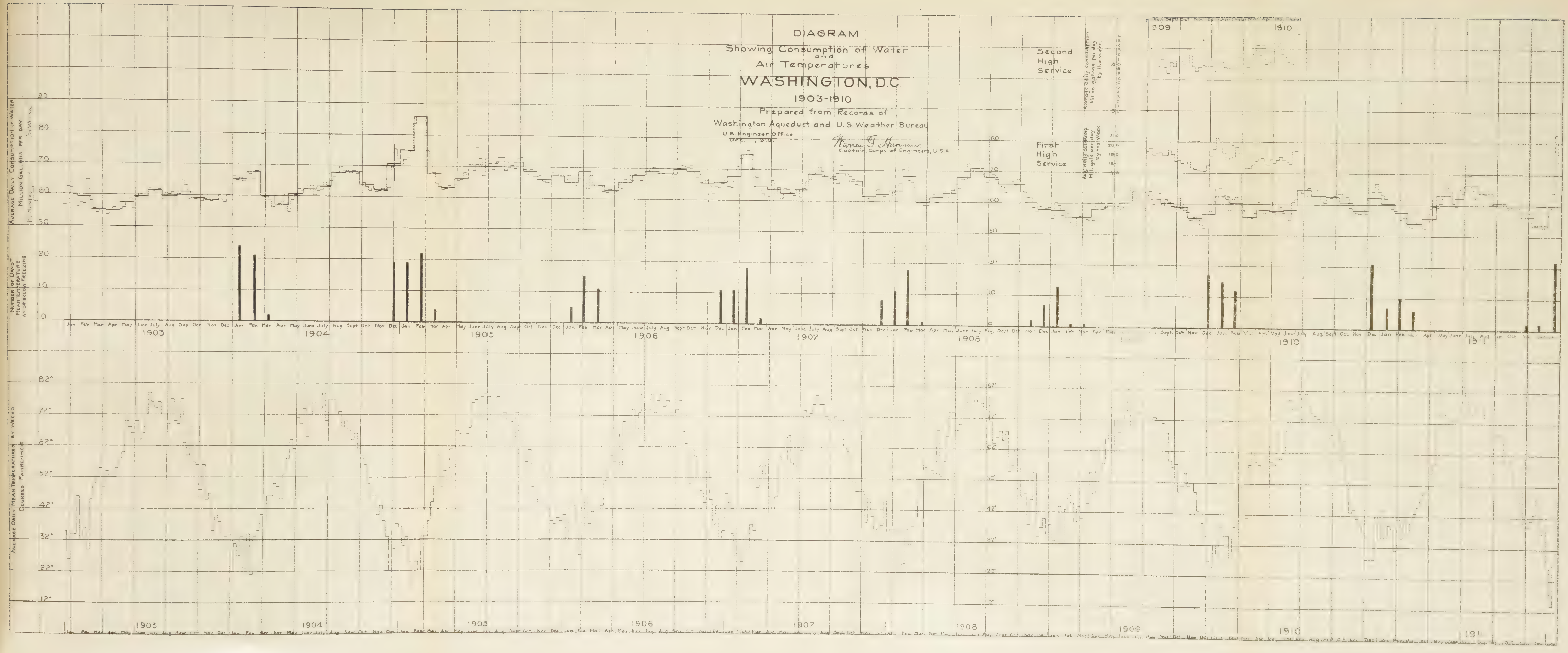


DIAGRAM
Showing Consumption of Water and Air Temperatures
WASHINGTON, D.C.
1903-1910
Prepared from Records of
Washington Aqueduct and U.S. Weather Bureau
U.S. Engineer Office
Dec. , 1910.
Warren T. Hannum,
Captain, Corps of Engineers, U.S.A.
Second High Service
First High Service
Average Daily Consumption of Water Million Gallons per Day
Number of Days Mean Temperature at or below Freezing
Average Daily Mean Temperatures by Weeks Degrees Fahrenheit
1903
1904
1905
1906
1907
1908
1909
1910

of equal severity before 1906, is believed to be due to the effect of the installation of meters.

Although a meter will not entirely eliminate the waste due to the running off of water to prevent freezing of water in service pipes, it will greatly reduce such waste, because care will be taken to waste only sufficient to prevent freezing. The records of metered cities seem to show tendencies to increased consumption in cold weather. To determine, if possible, the relative amount wasted from metered and unmetered services the curves of consumption of water for the first and second high services of the city were plotted on the chart for the fiscal year 1910. The second high service is almost entirely metered, while the first high service is nearly unmetered. Comparison indicates that the waste from the average metered service is about one-fourth as much as from the average unmetered service.

As there were in November, 1904, about 51,100 unmetered services and 2,000 metered services, granting that the average metered service wastes one-fourth as much as the average unmetered service, the water wasted in that year was the same as would have been wasted from $\left(51{,}000 \text{ plus } \frac{2{,}000}{4}\right)$ 51,600 unmetered services. As there are now about 47,532 unmetered services and 16,915 metered services, the water to be wasted would be the same as that wasted from $\left(47{,}532 \text{ plus } \frac{16{,}915}{4}\right)$ 51,761 unmetered services. Therefore, the maximum waste to be expected this winter, if the cold should be equally as severe as that of 1904-1905, would be $\frac{51{,}761}{51{,}600}$ of the amount of the waste that occurred in February, 1905, or twenty-eight million gallons daily, creating a maximum rate of consumption in February, 1911, of about (58 plus 28) eighty-six million gallons daily. As the pressures have generally been increased since 1905, the increase would probably be greater.

As the records show that periods of cold weather similar to the winter of 1904-1905 have occurred at irregular periods with an average interval of five or six years, it is not irrational to assume that another winter of equal severity is likely soon to occur again.

In February, 1905, if the cold weather and the high rate of consumption had continued five days longer, the reserve supply of water in the reservoirs would have been exhausted. Thereafter

demands in excess of the capacity, or an interruption of the supply at any point on the conduit, would have meant no less than a shortage of supply to the city. It is needless to say such a condition was serious and a recurrence should be carefully avoided; first, by increasing the maximum flow of water to the city's distribution system, or second, by greatly reducing the maximum probable waste in periods of very low temperatures, or third, by greatly reducing the normal consumption of water by detecting and stopping wastage through permanent leaks. To follow the first method would require the construction of another aqueduct in order to obtain any material relief. Such a course was not recommended by the officer in charge of the Washington Aqueduct, provided universal metering should be promptly established. From the records there appears no positive indication that the second has been accomplished. The third has been accomplished to a slight extent, as the rate of consumption of water in November, 1910, was four million gallons daily less than in November, 1904, and therefore there is an apparent margin of safety of four million gallons daily in case of a recurrence of abnormally low temperatures similar and equal to those of the winter of 1904-1905. But, unless the prevention of waste of water progresses at a more rapid rate than during the past year this small margin will be eliminated within a year or two, for notwithstanding the waste discovered and corrected by the pitometer division of the Engineer Department of the District, during the past year the normal consumption as indicated by the rate for the month of November has increased 2.6 million gallons, the average increase in the rate of consumption during the first five months of the fiscal year 1911 over the corresponding period in the fiscal year 1910 having been 1.7 million gallons daily.

The possibility of a shortage of the water supply within a few years can not be overlooked. The present conditions are very slightly better than during the fall of 1904. As five years will be required to secure the necessary lands and construct a new aqueduct, the normal consumption of water and the winter peak must be reduced to such an extent that when it shall appear to be no longer possible to prevent the natural annual increase in the normal consumption of water, due to the growth of the city, at least five years shall remain before the rate of consumption of water during a very severe winter or very warm summer would be likely to equal the maximum capacity of the aqueduct.

To endeavor to prevent the possibility of the demand for water exceeding the supply under any conditions by meeting the natural increase in demand for water by an equal or slightly greater annual reduction in waste is not a safe course, because, when it shall appear no longer possible to maintain the condition of equilibrium, sufficient time will not be available to complete a new aqueduct before the occurrence of a shortage of supply.

Although it was estimated in the report on the Investigations for the Increase of the Water Supply of the District of Columbia that with universal metering established in the city, the per capita consumption of water should be reduced to 135 gallons per day, and therefore the completion of a new aqueduct would not be necessary for the estimated population until 1930, it is not believed wise, in a matter as vital to the welfare of a city as its water supply, to accept such estimate as a fact, and therefore to merely attempt to gradually reduce the per capita consumption as the population increases, *but the estimates should be verified without delay by placing a meter on every water connection.*

In view, however, of the desire of the Commissioners of the District of Columbia to avoid as long as possible the expenditure of funds necessary for universal metering in order that money may be available for other urgent public improvements, it is believed that if meters should be installed approximately at the rate shown in the table below, the consumption of water will be kept within such limits that the danger of the demand exceeding the supply will be eliminated, and sufficient time will be available to complete a new aqueduct before the capacity of the present aqueduct shall be exceeded.

It has been estimated that the present aqueduct will furnish a supply sufficient for the city until 1930, provided the per capita consumption can be reduced to 135 gallons per day and provided the population of the city shall not exceed 460,000 in 1930. This estimate allows a maximum draft of 45 per cent in excess of the mean annual average, which will probably be reduced at least one-third when the universal metering shall have been established, thus operating to further postpone the date on which a new aqueduct will be necessary.

Assuming the correctness of the above estimate, there would be eighteen years beginning with the fiscal year 1913 in which to meter the 47,532 services unmetered at the present time, and thus accomplish universal metering, provided that all new services

were metered in the same year in which they would be placed. If the installation of meters on the present unmetered services was distributed evenly over the eighteen years, 2,640 meters a year would be installed in addition to meters on new services, which might be a safe method to follow if the above estimate were a certainty. As it is not a certainty, and as the necessary legislation for a new aqueduct should be secured in the sixth year preceding the year in which the additional supply will be necessary in order to allow five years for acquiring land and for construction, the plan adopted for universal metering should provide for the completion of the metering in 1924. Accordingly, the following table has been prepared, which provides for the installation, in the first three years, of the 15,840 meters that would be installed during the fiscal years 1924 to 1930, if meters should be placed at a uniform rate of 2,640 meters per annum during the period 1913 to 1930.

TABLE 6.—*Proposed Plan for Installation of Meters.*

Fiscal years.	*Number.*	*Remarks.*
1913	7,920 new meters.	In addition all new services should be metered in the year in which they are placed.
1914	7,920 new meters.	
1915	7,920 new meters.	
1916-1924	2,640 new meters each year.	

To install meters at the rate proposed would require the following expenditures, estimating the average cost of a meter in place at $15.00, and the average number of new water services per year about twenty-four hundred, namely, $155,000.00 per year for three years from the fiscal year 1913 to the fiscal year 1915, inclusive, and $75,000.00 per year for nine years from the fiscal year 1916 to the fiscal year 1924, inclusive.

The installation of a larger number of meters in the first three years is necessary in order to know the actual effect of metering as compared with the estimated effect, so that if the reduction in per capita consumption should be less than expected, or if conditions should change, making it necessary to have an additional supply sooner than anticipated, time would remain to accomplish the necessary work before the maximum demands of the city should exceed the limit of the present supply. After the proposed plan has been completed to the end of the second year, a study of the per capita consumption, population, and the extent of the winter peak as

compared with the peak consumption in previous winters of equal severity may indicate that the metering then remaining to be accomplished may be more uniformly distributed over the subsequent years, the object being at all times to keep the supply at least six years ahead of the consumption. When it appears no longer possible to accomplish this, it will be necessary to procure legislation to provide for an additional supply.

The urgency of action to reduce the consumption has probably not been apparent because no winter with a period of three weeks of continued low temperature in February preceded by other periods of low temperatures, as happened in 1905, has occurred since the latter date to indicate what the maximum draft under such conditions would be. In no winter since 1905 has the average daily consumption for any week exceeded by 18 per cent the average daily November consumption, whereas the maximum average daily consumption for any week in February, 1905, was 45 per cent in excess of the average daily consumption for the preceding month of November, which under present conditions is likely to occur again.

Unless a plan of metering similar to that outlined herein is adopted, it is not improbable that the aqueduct will within a few years be unable to furnish "the unfailing and abundant" supply of water for which it was designed, and which function it has been able to fulfill for fifty-one years.

ONE YEAR LATER.

Recently the Commissioners of the District of Columbia, acting under Act of Congress granting them power to charge private water takers in the District of Columbia not to exceed 30 cents a thousand gallons for water supplied them, have provided for increasing the water rates 33 1-3 per cent above the present rates. The new rates are to become effective July 1, 1912, and universal metering of private services is to be accomplished in approximately six years thereafter (July 1, 1918) by the expenditure of the additional revenues received under the new rates.

The increased rate for unmetered water service will have no effect on checking the waste of water, but if there be any effect on such service it will be to increase the waste by the desire of the user to get more if he must pay more. However, such services will be rapidly metered, so that any such tendency will be quickly offset by the desire of the metered consumer to check any waste of water.

The metered rates are to be increased from three cents for 100 cubic feet of water to four cents for 100 cubic feet. The minimum annual charge will be $4.50 for annual consumption of any amount below 7,500 cubic feet instead of 15,000 cubic feet as at present.

The reduction in the maximum amount allowed at the minimum rate will have a direct and very beneficial effect on the reduction of waste. Under the present allowance of 15,000 cubic feet, there are comparatively few metered domestic services charged more than the minimum, but under the new rate doubtless many will be charged in excess of the minimum, resulting in efforts, in most cases, to get the benefit of the minimum rate by saving that which now performs no useful service. Even under the new rates the maximum allowance at the minimum rate will permit each metered connection to draw off an average of nearly 21 cubic feet per day, or about 4 cubic feet per capita per day, a liberal allowance if only a reasonable amount of care is exercised in the use of the water, an allowance which will certainly not be a "menace to the public health," an argument which is always used by the opponents of the installation of meters, though fortunately less effectively than in the past. The allowance of 15,000 cubic feet, without additional charges above the minimum charge, has probably tended in some cases to increase the draft of water, and undoubtedly has not fully accomplished the object of installation of the meter, i. e., the checking of waste, by permitting a large draft of water in hot summers and cold winters and a comparatively small draft at other seasons. It might be found advisable to divide the annual allowance at the minimum rate into semi-annual or preferably quarterly allowances, in order to more effectually check the waste at times when the tendency to waste is greatest, although under the new annual allowance of 7,500 cubic feet the necessity will not be so evident as under the present allowance of 15,000 cubic feet.

Congress has begun to provide for the installation of meters on services to public puildings and grounds, and it is hoped that the installation will be continued and completed within the next few years so that universal metering will be accomplished by 1918 before which year the wisdom of the installation of meters will be fully demonstrated.

The accompanying diagram has been extended to include November, 1911. The additional information thus afforded does not

indicate that the installation of meters is any less urgent than a year ago.

FEBRUARY, 1912

Since submitting the paper on "The Adequacy of the Washington Water Supply" a period of low temperature has occurred by which the sensitiveness of the water system, the amount of variation in the demand for water by the city due to a period of freezing temperatures, has been tested. The diagram showing air temperatures and water consumptions has been extended to include this period, from which it will be noted that the consumption reached an amount not equalled since 1905 and also that the period of low temperatures and intensity of cold was unequalled since 1905, thus indicating, in a general way, that the deductions contained in the paper mentioned above regarding probable high consumptions approaching the capacity of the conduit, were not incorrect.

In order to examine conditions more closely the following comparative table relating to consumption of water is submitted:

Consumption.

(*a*) Minimum average weekly immediately preceding cold period.	(*b*) For preceding November. (Normal)	(*c*) Maximum average weekly during cold period.	Increase (*c*)	
			Over (*a*)	Over (*b*)
			Per cent	*Per cent*
1905—70 m.g.d.	62 m.g.d.	90 m.g.d.	29	46
1912—53 m.g.d.	56 m.g.d.	83 m.g.d.	57	48

By inspection of this table it will be evident that the system is more sensitive to cold waves under present conditions than it was six or seven years ago, requiring a greater increase of supply to meet the demand during a cold period. The conditions just passed which, on account of the shorter duration of the cold wave and the lack of a cold wave earlier in the season, did not form as severe a test of the adequacy of the supply as those of seven years ago, yet the demand at the end of the cold wave for a few days was nearly equal to the maximum capacity of the conduit.

It might be argued that the cold was more intense this year than in 1905. Although the daily mean temperature reached a lower figure than in 1905, this was true for only two days when the means were 0 and 4. The duration of freezing temperature was twenty-nine consecutive days in 1905 compared with fourteen consecutive days in January, 1912. In addition it will be noted that in

December, 1904, there occurred a short period of freezing temperatures causing a minimum average weekly rate of consumption preceding the cold wave of January and February, 1905, 13 per cent above the November (normal) average, whereas in December, 1911, there was no such period of freezing temperatures. Had such a condition occurred last December, or had the freezing temperatures continued seven consecutive days longer in January, the average weekly consumption would have exceeded ninety million gallons per day. Further, should a period of fourteen or more (perhaps fewer) consecutive days of freezing temperatures occur in February, 1912, the demand will again exceed the capacity of the conduit. Whether or not conditions would be serious would depend on the duration of the cold period and also upon the actual demands upon the system.

Prior to actual existence of a shortage of supply to meet the demand, and before all storage is exhausted above the hydraulic gradient for ninety-million gallon-day discharge through the conduit, it would probably be found necessary to admit unfiltered water to a part of the distribution system due to the probable failure of the pumps to lift a sufficient quantity of water to the filters to meet very high demands for a few days, when the lift would probably exceed 26 feet.

Navigation Companies vs. Water Power Users Sebago Lake, Maine

BY

Capt. Lewis M. Adams
Corps of Engineers

In 1820, the Cumberland and Oxford Canal Company was incorporated under charter from the State of Maine. Section 2 of the act of incorporation defines the powers granted and limits the route and terminii of the canal as follows: ''That said corporation shall have power to survey, lay out, make and forever maintain a canal or canals with a suitable number of locks to commence at the waters of Thomas Pond in Waterford in the county of Oxford, thence proceeding to Sebago Pond, and thence to the navigable waters of Fore River in Westbrook, county of Cumberland, following such direction and terminating at such place on said river as they may designate.'' About 1830 the canal was built, reaching from tide water to Lake Sebago and for the most part paralleling Presumpscot River, which is the outlet to the Atlantic from the lake. The fact may be noted that the canal company never built that portion of the canal from Thomas Pond to Sebago Lake; it did build a lock in Songo River establishing a route to Brandy and Long Ponds above, but apparently this construction was undertaken without legislative authority, and it was not until many years later that it acquired by purchase any right at the Songo Lock. Under its charter the corporation was also authorized ''to take and use the water of and from any pond or ponds, rivers, and other water courses for the purpose of supplying and maintaining said canal or canals.'' The charter gave no authority to use Songo River as part of the canal. The act of incorporation did not grant the right to have the water of Sebago Lake held at a sufficient height to ensure navigation on Songo River nor did it give the right to have the lake held above its natural level. It did grant to the canal company the right to take water from the lake for canal purposes and to erect a dam at Westcotts Falls to afford such a

supply. The charter did not undertake to confer any rights in respect to navigation on Lake Sebago or the Songo River, its sole purpose was evidently to provide for navigation from the lake to Portland, and from Thomas Pond to Lake Sebago. As previously noted the latter route was never opened.

The canal company took, or attempted to take, an existing dam at Westcotts Falls, at which point the canal to tidewater began, but there is no record that compensation was ever made to the water power users already operating below. If the dam was ever legally taken, obviously it could only be for the purposes stated in the act of incorporation. From the time the canal was built the mill owners always insisted upon their right to have the water, which this dam and the one which replaced it held back, flow down to them at all times when needed. This right was never denied by the canal company between 1830 and 1860. Waste gates were put into this outlet dam at Westcotts Falls by the power owners, and when a new dam was constructed in 1857 at the expense of the canal company and the power owners, the company paid one-third and the power owners two-thirds of its cost. The power owners opened the gates of this dam whenever they wished flowage, and in 1860 a suit was brought to prevent them from so doing. The court decided in favor of the power owners and their rights were never afterwards questioned. It would thus appear that even considered as a dam for the purpose of supplying water to that part of the canal from the lake to tidewater, the canal had no exclusive right therein for that purpose as against the mill owners' needs. In 1876, by judgment of court, the canal company was ousted of its privileges, and whatever rights it had in the canal reverted to the owner of the land, the predecessor in title to the Presumpscot Water Power Company. All rights of all parties to the storage and regulation of the waters of the lake were vested in the power company and to this power company the Cumberland and Oxford Canal Company executed a quit claim.

The top of the old dam at Westcotts Falls was at elevation 265.13; this old dam was rebuilt, in 1879, by the water power company with the new crest at elevation 270, under the following grant in its charter "to rebuild, raise, and extend the present dam upon said Westcott's falls, and said dams may be rebuilt or raised to the height of five feet above the present top of said dam and shall be maintained for the purpose of raising a head of water for the use of factories and for the purpose of supplying the City of


SEBAGO LAKE
AND
PRESUMPSCOT RIVER DRAINAGE BASIN
Scale of Miles
5 4 3 2 1 0 5
KEY
A - Songo Lock
B - Proposed Lock
C - Resevoir Dam (Westcott's Falls)
D - Cumberland Mills
North
OXFORD CO.
CUMBERLAND CO.
OXFORD CO.
ANDROSCOGGIN CO.
ANDROSCOGGIN CO.
CUMBERLAND CO.
THOMPSON LAKE
LONG LAKE
Brandy Pond
SEBAGO LAKE
STEAMBOAT ROUTE
LITTLE SEBAGO LAKE
MAINE CENTRAL R.R.
MAINE CENTRAL R.R.
Saco River
SEBAGO LAKE
P. & O.
PORTLAND & ROCHESTER R.R.
Presumpscot River
PORTLAND

Plate I.

Portland with water, for the purpose of storing up water in Lake Sebago and its tributary ponds, in order to increase and regulate, but not unnecessarily to obstruct the flow of water on Presumpscot River for the benefit of all the water powers and mill privileges on said river and for any other lawful purposes.''

The old Cumberland and Oxford Canal, with its twenty-two locks to tide water, was a very great work for its day and represented a supreme effort of the people at that time. The C. and O. Canal was operated for about forty years and greatly aided in the development of the lake country north of Portland. It was part of the up-State fur-traders' route and aided materially in the lumber industry. Its life ended with the building of the railroad in 1870; its financial record throughout its existence was a most precarious one.

The regular steamboat route is from Sebago Lake Village Landing at the foot of Sebago Lake up into the Songo River, then through Songo Lock up into Brandy and Long Ponds, reaching the small towns of Bridgeton, Harrison, and others farther back. Songo Lock is all that is alive of the old route to tide water to-day. The embankments and locks along the line of the Presumpscot River are fast becoming obliterated by the development of water powers and time's natural decay.

After the building of the new and higher dam, with crest at elevation 270 at the mouth of the lake, the mean level of the lake rose materially, and this condition was naturally taken advantage of by the boat companies who soon navigated Songo River at greater draft than they had before, and on building new steamers they were designed to draw more water than the old ones. It also seems true that the mean annual rainfall follows certain cycles, and the higher crest dam was built at a time when the annual rainfall for several succeeding years was above the average. When the natural adjustment of this excess occurred, the navigation interests were the first to suffer and make complaint. The power companies proceeded to draw on the reserve storage in Sebago Lake which they had secured through the rebuilding of the dam by the Presumpscot Water Power Company, and although the resulting loss of depth was not as serious as would have occurred had the new dam not been built and had the same withdrawals for power been made, still, the boat companies had grown to look upon the increase in depth as a permanent blessing only to be disturbed by the greed of the power users.

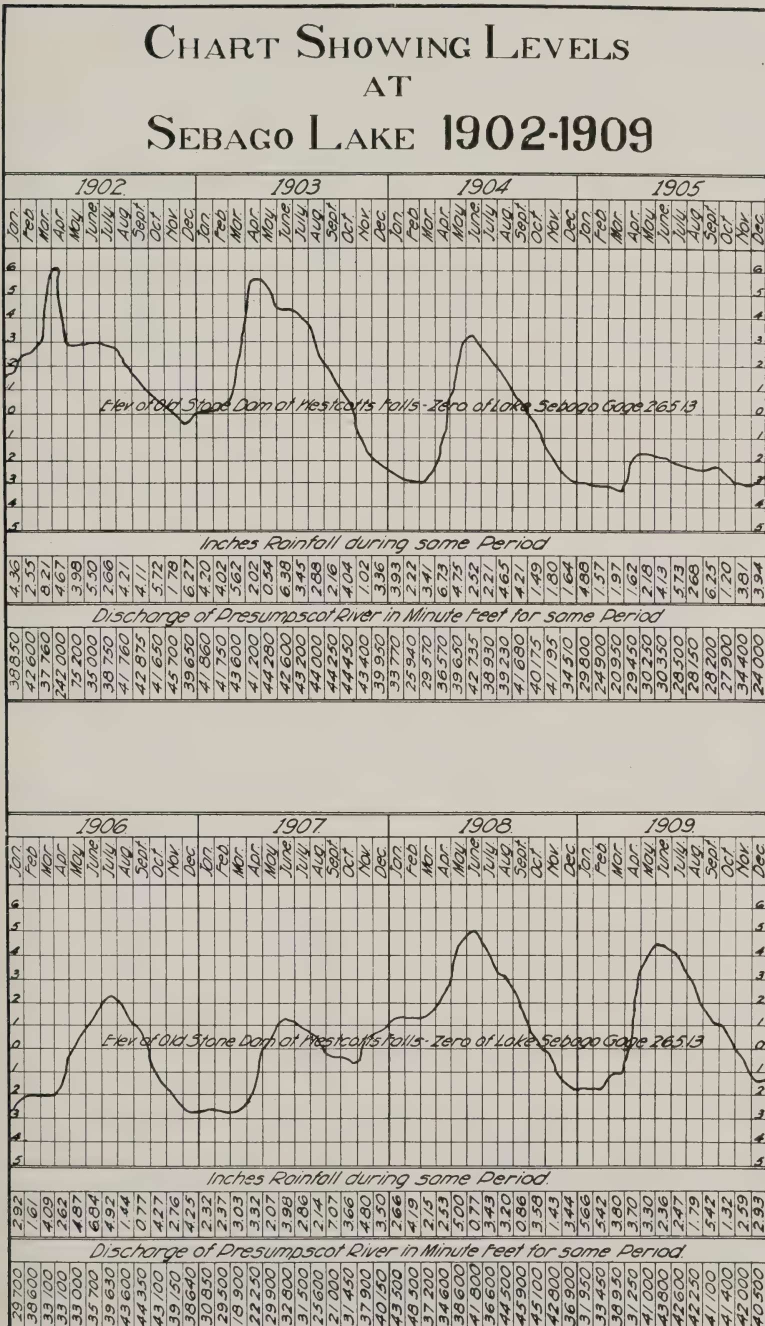


CHART SHOWING LEVELS
AT
SEBAGO LAKE 1902-1909
1902
1903
1904
1905
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
Elev. of Old Stone Dam at Westcotts Falls - Zero of Lake Sebago Gage 265.13
Inches Rainfall during same Period
4.36 2.55 8.21 4.67 3.98 5.50 2.66 4.21 4.11 5.72 1.78 6.27 4.20 4.02 5.62 2.02 0.54 6.38 3.45 2.88 2.16 4.04 1.02 3.36 3.93 2.22 3.41 6.73 4.75 2.52 2.21 4.65 4.21 1.49 1.80 1.64 4.88 1.57 1.97 1.62 2.18 4.13 5.73 2.68 6.25 1.20 3.81 3.94
Discharge of Presumpscot River in Minute Feet for same Period
38850 42600 37760 242000 75200 35000 38750 41760 42875 41650 45700 39650 41860 41750 43600 41200 44280 42600 43200 44000 44250 44450 43400 39950 33770 25940 29570 36570 39650 42735 38930 39230 41680 40175 41195 34510 29800 24900 20950 29450 30250 30350 28500 28150 28200 27900 34400 24000
1906
1907
1908
1909
Elev. of Old Stone Dam at Westcotts Falls - Zero of Lake Sebago Gage 265.13
Inches Rainfall during same Period.
2.92 1.61 4.09 2.62 4.87 6.84 4.92 1.44 0.77 4.27 2.76 4.25 2.32 2.37 3.03 3.32 2.07 3.98 2.86 2.14 7.07 3.66 4.80 3.50 2.66 4.19 2.15 2.53 5.00 0.77 3.43 3.20 0.86 3.58 1.43 3.44 5.66 5.42 3.80 3.70 3.30 2.36 2.47 1.79 5.42 1.32 2.59 2.93
Discharge of Presumpscot River in Minute Feet for same Period.
29700 38600 33100 33100 33000 35700 39630 43600 44350 43100 39150 38640 30850 29500 18900 22250 29900 32800 31500 25600 21000 31450 37900 40150 43500 48500 37200 34600 38600 41800 36600 44500 45900 45100 42800 36900 31950 33450 38950 31250 41000 43800 42600 42250 41100 41400 42000 40500

Fig. 1.

Whatever may have been the situation after the dam was rebuilt and the Presumpscot Water Power Company assumed control, it is certain that prior to that time the average level of the lake was lower, the periods of low water longer, and navigation of the lake more difficult than it has been since.

There was also chartered the Sebago Improvement Company, which bought all the rights and interests in Songo Lock and operated it according to its charter, which further contemplated the maintenance of navigation in Songo River. The discharge weir at Songo Lock was rebuilt at a higher elevation and this resulted in damage claims by the riparians on Long and Brandy ponds. The claims were all settled by the Sebago Improvement Company, and the result was greater depths for boats and more water for storage.

Scattered along the Presumpscot, from just below Sebago Lake nearly to tide water, are important industrial plants employing many thousands of operatives, all dependent to a degree on the power developments of the regulated stream.

Among these industries may be noted S. D. Warren & Co., paper manufacturers; Dana Warp Mills, carpet makers; Haskell Silk Company; Robinson Silk Manufacturing Co.; Androscoggin Pulp Co.; Portland Lighting and Power Co., and the Eastern Dynamite Company. It is not hard to see that interests dependent on these water powers would put up a strong fight to maintain their legal right to the use of the waters of Lake Sebago impounded by the reservoir dam built at their expense. Furthermore, it would be reasonable to assume that the fairest consideration would be accorded the power users, as it is already recognized that the future wealth of New England must lie in her water powers developed and undeveloped.

In November, 1910, those people allied with the navigation interests on Lake Sebago began a determined campaign for the establishment of a bench mark or level on the lake below which it would be unlawful to draw down the lake surface, and a bill was drafted to be presented as soon as the State legislature convened. This bill, if passed, would prohibit water being drawn from Lake Sebago when by so doing the lake would be lowered below a point 5 feet 4 inches above the level of the lower miter sill of Songo Lock, or what amounted to the same thing, below the height of the top of the old dam which existed prior to the incorporation of the Presumpscot Water Power Company. The top of this old dam, as

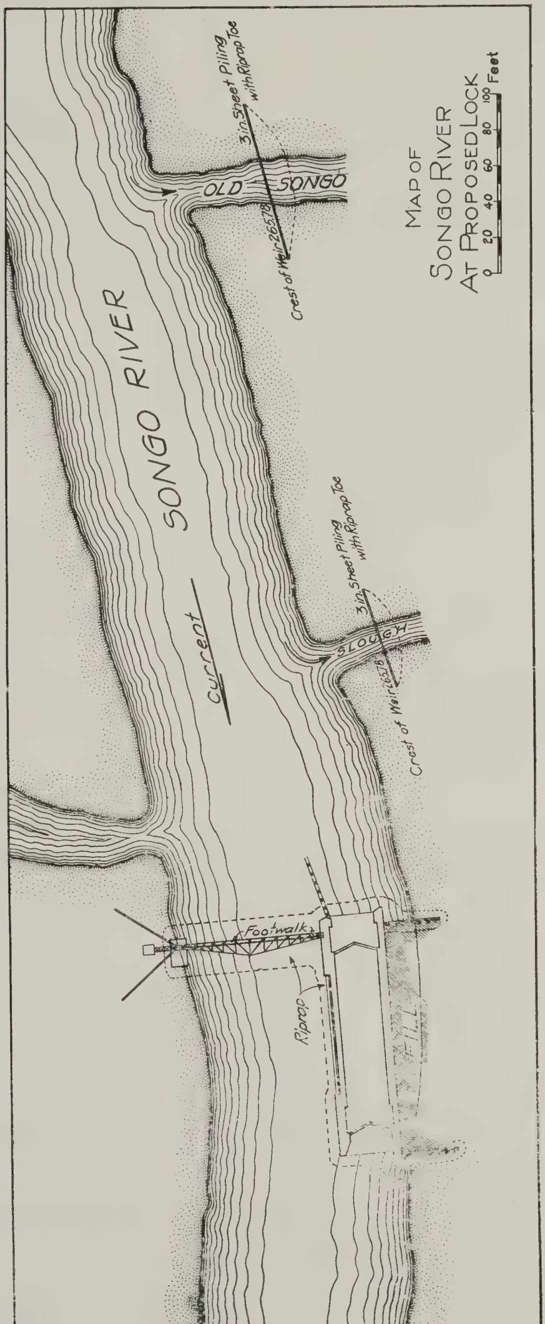

SONGO RIVER
OLD SONGO
3 in. Sheet Piling with Riprap Toe
Crest of Weir 265.78
Current
SLOUGH
3 in. Sheet Piling with Riprap Toe
Crest of Weir 265.78
Footwalk
Riprap
MAP OF
SONGO RIVER
AT PROPOSED LOCK
0 20 40 60 80 100 Feet

Plate II.

previously noted, was at elevation 265.13; the new dam having crest at elevation 270.

The effective capacity of the new dam to store surplus water is only 7½ feet, so that the passage of such a bill would deprive the power users of at least 2½ feet of storage or one-third of their total storage.

All that has been done by the Presumpscot Water Power Company and the Sebago Improvement Company has, without any doubt, resulted in the general betterment of navigation upon Sebago Lake, but the arbitrary establishment of the minimum level proposed would be equivalent to insuring relatively deep draft navigation, whatever might be the actual season's rainfall. It would be reasonable to suppose that the navigation interests should take their chances with the power users.

The bill, in its original form, would very likely have passed had not the power company headed it off. This company, with its intimate knowledge of the whole situation, immediately issued a pamphlet giving the facts in the case and refuting the claims of the opposition. Part of the data preceding is from this pamphlet.

Mr. John E. Warren, the present head of the firm of S. D. Warren & Company and a director in the power company, conceived the idea of securing a satisfactory depth in Songo River during the navigation season by the building of a movable dam, with lock, near the mouth of the Songo River at about the location indicated on the map.

As the engineers of the power company were not familiar with any type of movable dam, the writer's name was given the company and through this means became in a slight way identified with the project. It may be here noted that the power company has acquired the rights to storage and flowage on practically every pond in the Sebago Lake drainage basin. The most important of these are Long and Brandy ponds, with a storage area of 13.3 square miles, which is about one-fifth the area of the lake, which covers 67 square miles. Just below Brandy Pond is the old Songo Lock, which is still operated and has a small regulating weir beside it, to release the waters from the storage area above into Lake Sebago as needed.

In the fall, when the heaviest drafts are made on the lake for stored water, weirs at the outlets of all these ponds are opened successively and the storage descends to Sebago Lake, maintaining its level and replenishing the reservoir supply. Many of these

ponds have quite large storage areas of 2 or 3 square miles and in the spring the melting of the heavy winter snows usually fills them; and this stored water being held back for a period, tends to prevent any wastage over the reservoir dam at the foot of the lake. Were it not for this carefully planned conservation of the water, an excess of flow would occur in the Presumpscot each spring and much valuable flowage would be lost over the waste weirs at the various developments.

On December 28, 1910, an inspection of the works at the head of Lake Sebago made evident the following: Old Songo Lock was badly in need of repairs, and the discharge weir alongside was of too small capacity to act as an effective inflow regulator to Sebago Lake. The regimen of Songo River is quite unusual; its bed and banks are composed entirely of the softest fine sand mixed with silt. The geological formations would indicate that the entire basin of the Songo was brought down by the swiftly flowing Crooked River, which comes into the Songo only a hundred yards below Songo Lock. In the spring Crooked River carries as much as 25,000 second-feet discharge, and as this volume flows directly into the Songo, whose slope is quite flat, the Songo is necessarily subject to heavy spring floods. The bed of the Songo being of most unstable material, this has naturally resulted in the river making for itself a typically tortuous channel with bends closely resembling on a small scale the Greenville bends on the Mississippi. In some places the Songo has cut across the neck at former bends, and in other places it appears as if it was about to cut across with the next flood out of Crooked River. The mouth of the Songo has been improved by the addition of simple riprap jetties. These jetties were added when the river broke through to Lake Sebago to one side of its former outlet. The delta of the Songo is growing constantly and investigation showed this growth to extend the shore outwards by about 30 feet per year.

There is always plenty of depth over the navigable routes in Lake Sebago, and with the maintenance of the jetties a depth of 6 feet is available up the Songo for about half a mile. It was at about this point that Mr. John Warren, of the power company, thought it might be advisable to build a lock and movable dam. A fixed dam could not be seriously considered, as the continued slackwater in its upper pool would cause such heavy deposits of river sand that navigation would soon be interfered with. A movable dam would only have to be kept raised about three or four months

of each year, and the accumulations for this period would be flushed down the channel and out into the deeper water of the lake at each spring flood.

After a careful inspection of the entire Songo River and a consideration of available foundations, the most favorable site for a lock and movable dam was indicated and the sketch drawing and estimate accompanying this paper were prepared, together with a report on the situation in general.

Before proceeding further it might be of interest to call attention to the very effective system which the power company has for controlling the outflow from Sebago Lake.

The Presumpscot River in its improved condition is, according to the reports of the Geological Survey, one of the very best water power streams in the United States. The total water surface of its drainage basin is 97 square miles; the area of the drainage basin at the outlet of the lake is 420 square miles; and at the river mouth 600 square miles. The fall from the crest of the crib dam at Westcotts Falls to mean low tide is 270.0 feet in a distance of 21.65 miles, or an average of 12.47 feet per mile.

During the past few years several new developments have been made along the river, so that the only portion now unimproved is a fall of about 56 feet about 10 miles above Cumberland Mills, which is now about to be developed.

The tributaries of the Presumpscot are not of very much importance, but some of them are outlets of ponds, thus affording additional storage capacity for fall use. Probably nowhere in the United States is there a better example of the successful storage of water and regulation of the flow of a stream.

The flow from Sebago Lake has been constantly recorded since January, 1887, the discharge being computed from the openings in the reservoir dam gates. The discharge characteristic of these gates under varying head was determined and tabulated by Mr. Hiram F. Mills, the eminent hydraulic engineer of Lowell, Mass.

In 1904, a development was completed at Eel Weir Falls, a short distance below Sebago Lake, bringing the water directly from the reservoir dam by means of a canal; there is thereby obtained a direct head of about 40 feet at average lake level. The gates at the reservoir dam are maneuvered by an electric motor controlled from the development below at Eel Weir, but this development has necessitated a different method of recording the discharge at the dam. The water at Eel Weir drives three pairs of 33-inch Her-

cules turbines, and three Allen meters record the water used at each pair. These meters were rated at the Holyoke flume by an actual test on one of the pairs. The performance of the wheels and of the recording meters has been checked by careful current meter readings in the canal and by Prony brake tests on the wheels, combined with metering of the generator output. Recording wattmeters give a constant record of the dynamos in this station and the ratio between these readings and the Allen meter records furnish a good check on the latter. It is usually desired to keep a constant flow through the outlet canal, and when the demands for power are not sufficient to utilize the entire flow through the wheels at the first development, the excess of water is run off through a pair of regulating gates, a record of the openings of these gates being kept and the flow computed from a coefficient determined by current meter tests. The flow from the lake may at times be greater than it is safe to carry through the canal, and at such times it will be necessary to draw off part of the water through the old regulating gates in the reservoir dam. This excess water will not necessarily be wasted, as the other developments at lower head farther down the river would take it through their wheels.

A continuous record of the level of Sebago Lake has been kept since January, 1872. The lake fills rapidly after March 1, attaining its maximum height between the middle of April and June 1, and then gradually subsides as the stored water is withdrawn for power until a minimum stage is reached—sometimes in the autumn, but usually in early winter.

The remainder of this article contains the substance of a report which was submitted at a meeting of the directors of the Presumpscot Water Power Company. The substance of the suggestions and conclusions reached in the report were followed by the power company. A compromise bill was passed by the State legislature, satisfactory to both parties, which required the Sebago Improvement Company to rebuild the Songo Lock and Weir and also required that a navigable depth be maintained from Sebago Lake to Brandy and Long ponds.

The following depths are to be available:

Not less than 4 feet 6 inches from May 1 to June 15.

Not less than 5 feet 4 inches from June 15 to September 15.

Not less than 4 feet 6 inches from September 15 to November 1.

The original bill which it was intended to introduce on behalf of the navigation interests was so worded that, if passed, it would

have required the power users to maintain a navigable depth over the route from the foot of Lake Sebago up to the north end of Long Pond, and it would at the same time have arbitrarily established a bench mark on Lake Sebago at elevation 265.13; this would have uselessly prevented the power users from securing part of the required depth by lowering the lower sill of Songo Lock. The suggestion to lower this lower-gate sill by 18 inches was carried out on rebuilding the old lock in the fall of 1911, and with a small amount of easy hydraulic dredging and bank protection the requirements of the new bill can be met.

A careful study was made of all feasible schemes to maintain a navigable depth at the head of Sebago Lake, up the Songo River and into Songo Lock, and at the same time to secure the Presumpscot Water Power Company in the use of the waters of Lake Sebago for power purposes on the Presumpscot River. There appeared to be four alternatives to effect this:

First. By raising the crest of the reservoir dam at Westcotts Falls.

It seemed possible to raise the crest of the reservoir dam from elevation 270 to a new elevation of about 273, obtaining thereby somewhat greater depth in the lake and in Songo River, the year around.

Further study showed this project to be open to most serious objections, among which were the almost prohibitive cost of purchasing the new additional flowage rights from riparians and the deprivation of the upstream developments of their usual flowage while the lake was gaining its new level.

An inspection of Chart of Levels of Lake Sebago, 1902-09, shows that if this method of securing a 6-foot navigable depth had been adopted for 1903, the conditions of inflow for that spring would have brought the lake level up to a bit over 269 at the opening of navigation on May 1, although the withdrawals for power in the meantime were rather above normal. This elevation of 269 is taken arbitrarily, because a study of the river profile gives reason to suppose that the bed of the Songo would silt up to an elevation of about 263 on the increased depths due to backwater from the higher lake levels. The lake level could have been maintained at or above 269 for the navigation season that year without much difficulty, and the normal withdrawals for power could have been made in the meantime. No water would have gone to waste over the new

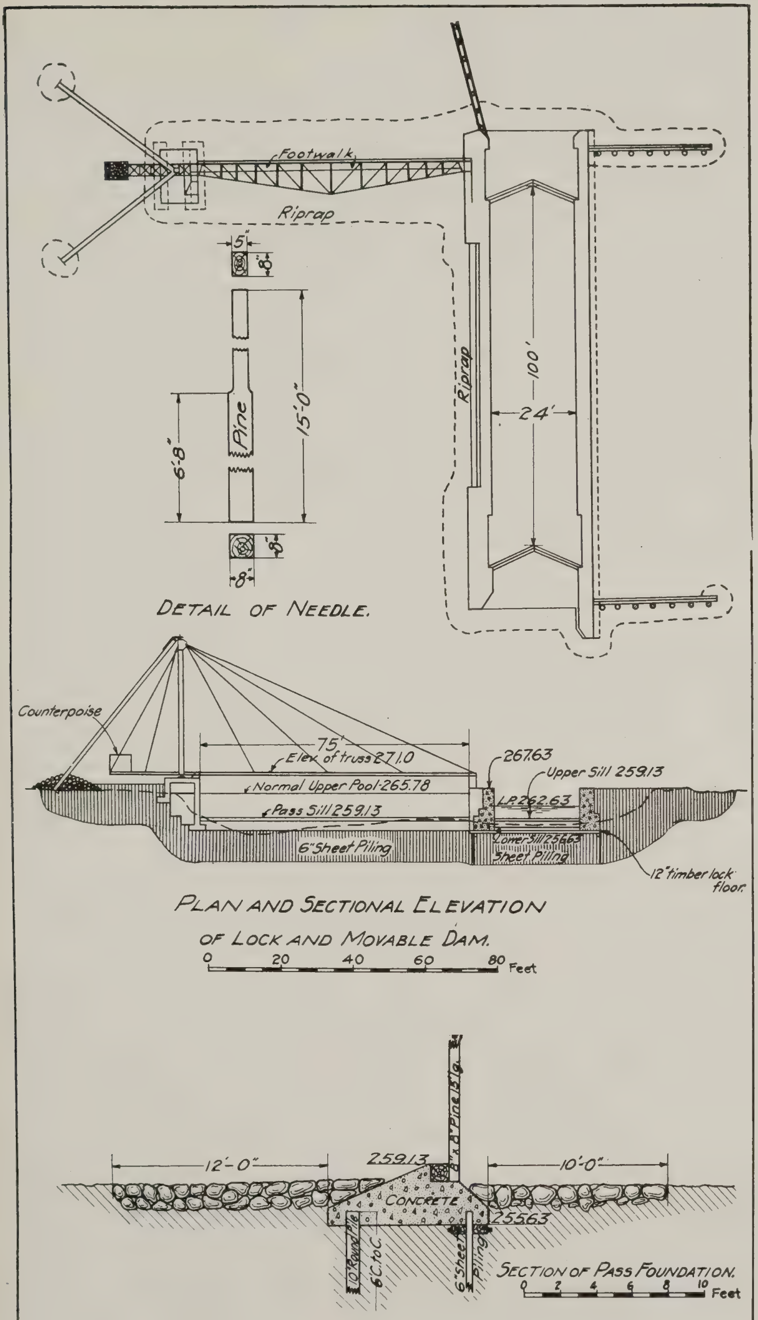

Footwalk
Riprap
Riprap
5"
8"
15'-0"
6'-8"
Pine
8"
8"
100'
24'
DETAIL OF NEEDLE.
Counterpoise
75'
Elev. of truss 271.0
Normal Upper Pool 265.78
Pass Sill 259.13
6" Sheet Piling
267.63
Upper Sill 259.13
L.P. 262.63
Lower Sill 256.63
Sheet Piling
12" timber lock floor.
PLAN AND SECTIONAL ELEVATION OF LOCK AND MOVABLE DAM.
0 20 40 60 80 Feet
12'-0"
259.13
10'-0"
CONCRETE
255.63
10" Round Pile
6'C. to C.
6" Sheet Piling
SECTION OF PASS FOUNDATION.
0 2 4 6 8 10 Feet

Fig. 2.

spillway, due to excess of inflow over usable outflow during the navigation period.

Next consider the project as having been put in effect in the year 1904. The chart shows that if about January 15, 1904, the reservoir gates had been closed, the lake would have risen to about 269 by May 1, 1904. From May 1 to June 10, it would have been necessary to withdraw about 38,000 minute feet for power to avoid having some of the storage go to waste about June 10, when under the new conditions the lake level would have just reached the spillway. The lake would not have fallen below 269 until after September 15, on the withdrawals that were actually made for power that season.

To have put this project into effect for 1905, we find that the lake level would have reached an elevation of 269 on May 1 that year if the outflow gates had been closed for the entire period subsequent to about December 1, 1904. To have held the lake to this level until the close of navigation would have required a reduction in the outflow to the wheels from about 500 second feet to about 325 second feet for this entire period of six months.

To have put this project into effect for May 1, 1906, the gates could have been closed about January 1, 1906, when the lake level would have risen to 269 by May 1; with the withdrawals for power which actually were made after May 1, the spillway would have been almost topped on July 15, and it is probable that to avoid any possibility of this wastage, much heavier withdrawals would have been made as lake level rose above about 270. It is consequently evident that the greatest difficulties would be met with in the practical application of this solution of the navigation problem; it would be absolutely impossible to tell how much water could be drawn from the lake during the late winter and early spring and at the same time still meet the requirement that the level of the lake would rise to 269 on or before May 1. It is almost certain that some years the spring rains would be so delayed that even with the flowage through the gates in the reservoir dam greatly reduced it would be impossible to raise the lake surface to afford a navigable depth on May 1, and it is equally probable that in some years these delayed rains would come in such volume after May 1 that the dam would be overtopped even at elevation 273 and a large amount of valuable storage would be lost, due to lack of development to effectively use it.

It appeared that the outlay for this doubtful project would ap-

proach $150,000. The increased backwater from Lake Sebago would certainly cause the Songo River to silt up an indeterminate additional amount, and to maintain the required navigable depth would necessitate intermittent dredging if at any time during the navigation season the lake level fell below about 269, as new bars would probably form on the increased backwater in the Songo River to at least elevation 263. At present there are six small bars in the river which project a foot or less above lower Songo Lock sill at elevation 259.78.

Second. By means of bank protection and dredging.

The adoption of this method would call for the lowering of lower Songo Lock sill about 18 inches when the excessive repairs would be undertaken at that lock. This would provide the required navigable depth, with a small amount of dredging and bank protection. The adoption of this project indicated quite an outlay; first, about $15,000 for thorough repairs at Songo Lock, including lowering its downstream sill 1½ feet to elevation 258.3; second, about $15,000 for a small 50-horsepower self-propelled hydraulic sand dredge and snagger with a maximum capacity of about 800 cubic yards per day of ten hours. The material to be dredged is fine smooth sand and ideal for such a plant. To maintain this dredge for five months per year would mean an annual expenditure of about $3,000, including depreciation. In addition, an initial expenditure of about $5,000, with $500 per year, could be very profitably expended for bank protection. This bank protection, if properly executed, would render but very little future dredging necessary.

The money involved in this project would thus be approximately:

Repairs to lock	$15,000
Dredge	15,000
Immediate bank protection	5,000
$3,500 per year at 5 per cent	70,000
Total	$105,000

Third. By means of an additional low lift lock and movable dam in the mouth of the Songo River.

The sketch drawings and estimate for this project accompany this paper. They show a lock 24 by 100 feet, with upper sill at 259.13, same as the navigable pass sill; lower lock sill at 256.63. The maximum lift at this lock for 6-foot navigation would be 3.15 feet. It could be used for 5-foot navigation if the steamboats did not demand more than 5 feet, and the extra foot of water could be

drawn off for power. The pass is closed by 8 by 8 inch wooden needles when the level of the lake falls below 263.63, thereafter all boats would go through the lock to upper pool.

To take care of any excessive discharge out of Songo River, as many of the needles can be removed as desired until all are gone, and the supporting truss is swung back, when the lock gates can be opened and the river runs by in open channel.

This lock could be built within the estimated cost of $20,763.00. Its maintenance and upkeep would amount to about $2,500 per year.

Assuming no expenditure for acquisition of land, the money involved in this project would thus be:

To rebuild Songo Lock	$15,000
To build lock and movable dam	20,763
To protect banks and build weirs	8,000
Upkeep, $2,500 at 5 per cent	50,000
Total	$93,763

The construction of a lock at the mouth of such a river as the Songo would involve a large element of risk. With its very unstable bed, soft bottom and banks, it would always threaten the existence of a lock built in one channel of its delta. A constant danger would exist that the river would break out to the lake at the bends above the lock, as the low soft banks are constantly giving way to the current and any stricture placed at the river mouth would increase the danger. Such an accident might occur in a spring flood and would leave the lock useless and buried in sand far to one side of the new channel.

Fourth. To maintain the lake level at or above 265.13

To be guaranteed during a navigation season beginning May 1 and ending November 1, and providing liquidated damages or even a penalty for each day during the navigation season that the level falls below 265.13. If this penalty were made sufficiently large, it might result in the boat companies compromising and taking upon themselves the rest of the burden; the Sebago Improvement Company might still operate the Songo Lock and Weir, but would no longer be responsible for the silt out of Crooked River or the maintenance of a navigable depth in Songo River. It would appear that the behavior of the Songo River in silting up, and cutting its banks at the bends, is to continue to be the sore spot in the whole situation, and if the navigation interests were themselves responsible they would very soon solve the problem by reducing

the draft of their small steamers, making them of the stern paddle wheel or tunnel type instead of the deeper draft screw propeller model.

New conditions, later developments, and business considerations, made it unwise to carry out in detail any single one of the four alternative projects outlined above. After a sharp contest between the contending parties the power company agreed to maintain navigable depths in Songo River as previously noted, also to rebuild and maintain Songo Lock. This agreement was enacted into law at the 1910-1911 session of the Maine State legislature. In pur-

ESTIMATED COST OF LOCK AND MOVABLE DAM PROJECT.

KIND OF MATERIAL	LOCK	DAM	ABUTM'T.	UPPER GUARDWALL	LOCK GATES	TRUSS	UNIT PRICE	AMOUNT
Concrete	15857	205	1150	—	—	—	$.40/cu.ft.	$6884.80
Excavation	15000	2000	1600	—	—	—	.04/cu.ft.	744.00
Fill	11800	600	200	—	—	—	.03/cu.ft.	378.00
Riprap	2500	3000	600	—	—	—	.10/cu.ft.	610.00
Oak Timber	sills 720	900	—	—	—	1600	80.00	257.60
Round Piling (lin. ft.)	400	260	—	—	—	—	.60	396.00
Sheet Piling	24300	4500	2700	—	—	—	60.00	1890.00
Iron	4000	2000	2000	100	—	8000	.08	1288.00
Hemlock INCLUDES GUARD WALL	67500	—	—	—	—	—	20.00	1350.00
Timber & Iron (Sq. ft.)	—	—	—	—	588	—	1.75	1029.00
Pine	—	needles 9000	—	—	—	—	25.00	225.00
Total	—	—	—	—	—	—	—	15052.40
6-ft. Earth Cofferdam (450')	—	—	—	—	—	—	3.00	1350.00
Removing Cofferdam (450')	—	—	—	—	—	—	2.00	900.00
Total	—	—	—	—	—	—	—	17302.40
Superintendence, Contingencies & Engineering, 20%	—	—	—	—	—	—	—	3460.48
Total	—	—	—	—	—	—	—	20762.88
Protecting Caving Banks above & Weirs across Old Songo & Slough	—	—	—	—	—	—	—	8000.00
Grand Total	—	—	—	—	—	—	—	28762.88

Fig. 3. Detailed cost of lock and dam. (See second and third items in preceding total estimate of cost.)

suance of this agreement, the power company has built and is maintaining a hydraulic dredge and has reconstructed old Songo Lock with large dimensions and with lower miter sill lowered $1\frac{1}{2}$ feet. This latter modification allows the maximum power withdrawals from the lake and at the same time permits the steamboats to pass the new lock on minimum lake levels without difficulty.

Furthermore, it would appear that there is no great advantage, and some decided disadvantage,in making heavy power withdrawals during the period from May 1 to November 1. The auxiliary steam plants at the mills will give a higher efficiency if worked

more during the hot summer weather, and as a very large amount of steam is required for heating purposes in the winter, it would seem wise to delay lowering the lake level by heavy withdrawals until cold weather in the fall of the year, then the water power development could be increased very effectively and carry a large proportion of the heavy winter load.

From Water Supply Paper No. 241, 1907-1908. U. S. Geological Survey.

Data furnished from records at Eel Weir Development. Daily discharge, in second-feet, of Presumpscot River at outlet of Sebago Lake, Me., for 1907.

	Jan.	Feb.	Mar.	Apr.	May.	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1907												
1	520	517	333	317	417	550	550	500	*263	407	540	*259
2	522	522	333	267	417	*417	547	488	275	405	540	677
3	522	*417	*228	267	417	550	543	463	275	463	*225	670
4	522	522	350	267	408	550	368	*333	275	552	542	672
5	522	522	350	285	*282	547	570	427	275	448	542	672
6	*382	522	350	272	417	550	493	433	275	*215	542	673
7	522	522	350	*173	417	550	*385	427	275	542	542	647
8	522	522	350	272	417	550	547	432	*272	542	545	*272
9	500	522	350	290	417	*375	548	427	333	542	545	668
10	515	*335	*225	328	417	550	545	430	337	542	*398	665
11	522	518	345	418	417	550	547	*370	335	542	685	670
12	513	518	345	417	*342	550	547	437	337	542	686	670
13	*393	517	345	418	548	552	547	437	337	*231	684	671
14	522	517	342	*317	548	552	*410	437	337	543	664	663
15	522	508	283	418	550	552	540	450	*308	540	674	*438
16	522	517	267	420	550	*367	538	435	337	540	661	672
17	522	*325	*192	417	552	552	548	435	357	540	*332	672
18	522	517	267	418	552	552	553	*347	433	542	691	677
19	522	517	267	415	*350	552	588	433	373	542	680	670
20	*417	517	267	400	552	552	593	433	408	*270	678	670
21	522	517	267	*300	552	552	*308	443	400	538	677	680
22	517	515	267	417	550	552	500	433	*258	538	672	*268
23	517	515	267	418	550	*439	495	433	387	540	647	677
24	520	*263	*208	417	553	550	497	433	382	540	*433	677
25	522	333	267	427	551	547	503	*373	393	542	670	675
26	522	433	283	425	*445	520	503	412	405	502	668	662
27	*427	333	325	413	550	513	493	373	410	*268	660	672
28	522	333	333	*287	552	540	*408	373	402	540	660	668
29	522	----	333	415	550	535	500	373	*160	540	672	*272
30	522	----	333	417	485	*433	500	373	403	553	663	667
31	522	----	*167	----	550	----	493	373	----	540	----	667

*Sundays.

John Newton*

John Newton was born in Norfolk, Va., August 24, 1823, and graduated second in his class at the United States Military Academy July 1, 1842, on which date he was assigned to the Corps of Engineers.

After serving one year as Assistant to the Board of Engineers, he was detailed for duty at West Point, first as Assistant Professor and later as Principal Assistant Professor of Engineering, being relieved from this last duty in July, 1846. He then became Assistant Engineer in the construction of Fort Warren, Boston Harbor, Fort Trumbull, New London Harbor, and as Superintending Engineer of Construction of Fort Wayne, Michigan, and Forts Porter, Niagara, and Ontario, New York, until 1852.

His work then took him to the Atlantic Coast, where he made surveys for breakwaters in Maine, and later for the improvement of the St. Johns River and other localities in Florida. During the years 1853-1856 he was engaged in work on lighthouses, in the trial and inspection of a dredge, and on several commissions for the improvement of the St. Johns River and other places in and around Florida, and particularly as a member of a special board to select sites and prepare projects for coast defenses in Alabama, Mississippi and Texas.

He was Chief Engineer of the Utah Expedition in 1858, and from that time until 1861 was engaged in fortification work at Sandy Hook and Fort Hamilton, New York. He served throughout the War of the Rebellion from 1861 to 1866, starting in as Chief Engineer of the Department of Pennsylvania, during which service he took part in one of the early battles of the War, that at Falling Waters, Virginia, June 30, 1861. Following this duty he became Chief Engineer of the Department of the Shenandoah and, later, Assistant Engineer in the Construction of the Defenses at Washington, D. C., where he was also in command of a brigade to man those defenses. He was made a Brigadier-General of Volunteers in September, 1861, and took part in the Peninsular Campaign in

*See frontispiece.

1862, being particularly engaged in the Battle of Gaines Mill. In September of that year he commanded a brigade in the Maryland Campaign of the Army of the Potomac and was severely engaged first in the Battle of South Mountain and, later, at Antietam, where he greatly distinguished himself. His services continued with the Army of the Potomac, where he commanded a division in the Rappahannock Campaign and in the Battle of Fredericksburg, in December, 1862. In the Chancellorsville Campaign, May, 1863, he took part in the storming of Maryre Heights and the Battle of Salem, the only places where the Union forces achieved any real success.

He was with the Army of the Potomac when that army moved into Maryland and Pennsylvania following Lee's second invasion of the North.

Following the death of Reynolds on July 1, Newton was given command of the First Corps and rendered valiant service in repelling Pickett's famous charge on the 3d.

In studying the biographies of Civil War commanders, one can not fail to be impressed with the great service rendered the Union cause by Engineer officers in command of troops. How great was this service is well shown by the Battle of Gettysburg, the turning point of the war, where the commanding general, two of the seven corps commanders, one division commander, and two regimental commanders were Engineer officers, besides General Warren, Chief Engineer of the Army and hero of Little Round Top, and W. F. Smith, commander of a division in the pursuit of Lee.

Gen. Newton continued in command of the First Corps, Army of the Potomac, until its reorganization in 1864, taking an active part in the Rapidan Campaign during the preceding October, November, and December.

When the Army was reorganized, he was sent to join the Army of the Cumberland, where he commanded a division in Sherman's campaign against Atlanta, taking part in the battles at Dalton, Ressacca, Adairsville, Dallas, Pine Mountain, Kennesaw, Passage of the Chattahoochee River, Peach Tree Creek, Siege of Atlanta, assault of the enemy's entrenchments at Jonesboro, Battle of Lovejoys Station and in the final occupation of Atlanta, September, 1864.

He assumed command in October, 1864, of the District of Key West and Tortugas, Florida, taking part in the action at the natural bridge near St. Marks, Florida, in March, 1865. Follow-

ing this duty he commanded the State of Florida, the District of Middle Florida, and sub-districts of West Florida and Key West until the fall of 1865, and the district of Middle, Southern, and Western Florida until January 24, 1866.

He was mustered out of the Volunteer Service January 15, 1866, after which he resumed his work in the Corps of Engineers, where he had reached the grade of lieutenant-colonel. He was stationed in New York City from 1866 to 1884, and had charge there of many important works. During this time the blowing up of the dangerous rocks in the entrance to the East River from Long Island Sound took place, the most famous being flood rock in Hell Gate, which was honeycombed with galleries and charged with over two hundred and eighty thousand pounds of explosives, mostly rack-a-rock. This work, bold and gigantic in conception, proved a very great success and highly creditable to General Newton.

In March, 1884, he was made Chief of Engineers, and after two and one-half years service in that capacity was retired at his own request in August, 1886. Immediately after retirement he was appointed Commissioner of Public Works of New York City where he remained about two years, leaving that work to become president of the Panama Railway, a position he held at the time of his death, May 1, 1895.

His advancement in the Corps of Engineers was as follows: Second Lieutenant, 1842; First Lieutenant, 1852; Captain, 1856, for fourteen years continuous service, and Major, 1861. He reached the grade of Lieutenant-Colonel, Corps of Engineers, December 28, 1865, and was mustered out of the Volunteer Service January 15, 1866.

As previously noted, he was appointed Brigadier-General, U. S. Volunteers, in the fall of 1861, which grade he retained throughout the War except the period March, 1863, to April, 1864, when he was a Major-General, U. S. V.

He received the following brevets for conspicuous services at various times during the war:

Lieutenant-Colonel, U. S. A., "for gallant and meritorious service at the Battle of Antietam;" Colonel, U. S. A., "for gallant and meritorious service at the Battle of Gettysburg;" Brigadier-General, U. S. A., "for gallant and meritorious service at the Battle of Peach Tree Creek in the campaign against Atlanta;" Major-General, U. S. V., "for gallant and meritorious service during the

Rebellion;'' and finally Major-General, U. S. A., ''for gallant and meritorious service in the field during the Rebellion.''

A reading of the above very brief account of the services of General Newton shows the ability and versatility of the man, and likewise the value of a broad education and training such as is afforded by the work of the Corps of Engineers. Surely it is a record to be proud of and yet it differs only in details from that of many Engineer officers who graduated from the Military Academy in the twenty-five years previous to the breaking out of the Civil War.—A. A. F.

A Two-Company Infantry Redoubt

BY

Capt. L. V. FRAZIER*
Corps of Engineers

The redoubt was built at Fort Sam Houston, Tex., by Company K, Third Battalion of Engineers, beginning April 19, and continuing intermittently until July 10, when the work was completed. Work was carried on only during the forenoons.

The soil where the redoubt was constructed varied in different parts of the site. The entire site was covered by a layer of gumbo, varying in depth from about 2½ feet to the full depth of 6½ feet. In some parts this was underlaid by a compact layer of large gravel, this in turn covering a bed of white clay mixed with gravel. The nature of the soil made excavation quite a task, as none of the material could be removed with a shovel without first being loosened with a pick.

Weather conditions were unfavorable for work during the last two-thirds of the construction, on account of the heat.

The method pursued in excavating the main trench was to assign a non-commissioned officer permanently to a section of the trench about 30 feet long, defined by traverses, and to give the same men for his detail each day as far as possible. This was found to work very well, as it fixed the responsibility for the section and, incidentally, permitted a comparison of the ability of the non-commissioned officers.

Excavation of the earth in this part of the work was done entirely with the pick and shovel, as the traverses did not permit the use of the plow and scraper, and, on account of the thinness of the sheathing (1 inch), the trench had to be carefully dressed.

In constructing this trench, the excavation was carried to a depth of about 3 feet for the full width of the trench, and a trench about 2 feet to 2½ feet wide sunk at the back edge, where the sheathing was to be placed; following this the overhead 10 by 12 inch timbers,

*Commanding Company K, Third Battalion of Engineers.

the back sheathing, and a part of the overhead 6 by 8 inch timbers were put in position after which the excavation was continued, the earth being thrown out on the overhead timbers. When the excavation had progressed sufficiently, so it would not interfere with the work, the remaining 6 by 8 inch timbers were placed and the revetment started.

The amount of earth excavated from the main trench was about 735 cubic yards.

The number of man-hours for completion of the main trench was 4,163 for labor and 920 for supervision. In addition carpenter work, which consisted in sawing timbers and lumber to proper dimensions and assisting in putting in steps, amounted to 262 man-hours for labor and 91 for supervision. Figures for supervision do not indicate what was actually necessary, as all non-commissioned officers available were used as a matter of instruction.

Timbers and sheathing were placed by each section detail without the assistance of carpenters further than the sawing of the timbers, and even this was in some cases done by the section.

The revetment was of sand bags (grain bags being used) and was very satisfactory, although the grain bags are not as durable as some other form of bag.

The plow was used as deep as possible in the gorge trench for loosening the earth, but scrapers were not used on account of the narrowness of the trench.

The amount of material excavated from the gorge trench was 192 cubic yards.

The man-hours required were: Labor, 967; and supervision, 143. The plow was used for 2 hours.

The ditch was made 2½ feet deep, part of the earth being put in the glacis and the remainder in the parapet. Plows and scrapers were used for this excavation and the labor involved was as follows:

Team-hours: Plow, 62; drag scraper, 512; wheel scraper, 95. *Man-hours:* Labor for plow and scraper, 585; supervision for plow and scraper, 183; other labor, 307; supervision, 43.

The amount of excavation from the ditch was 2,314 cubic yards.

Amount of excavation for latrines, 37.7 cubic yards.

Labor, 332 man-hours; supervision, 55 man-hours.

The above includes placing timbers and back fill, as well as excavation.

The entanglement was constructed wholly of barbed wire, there being 1,180 square yards, of which about 350 square yards was

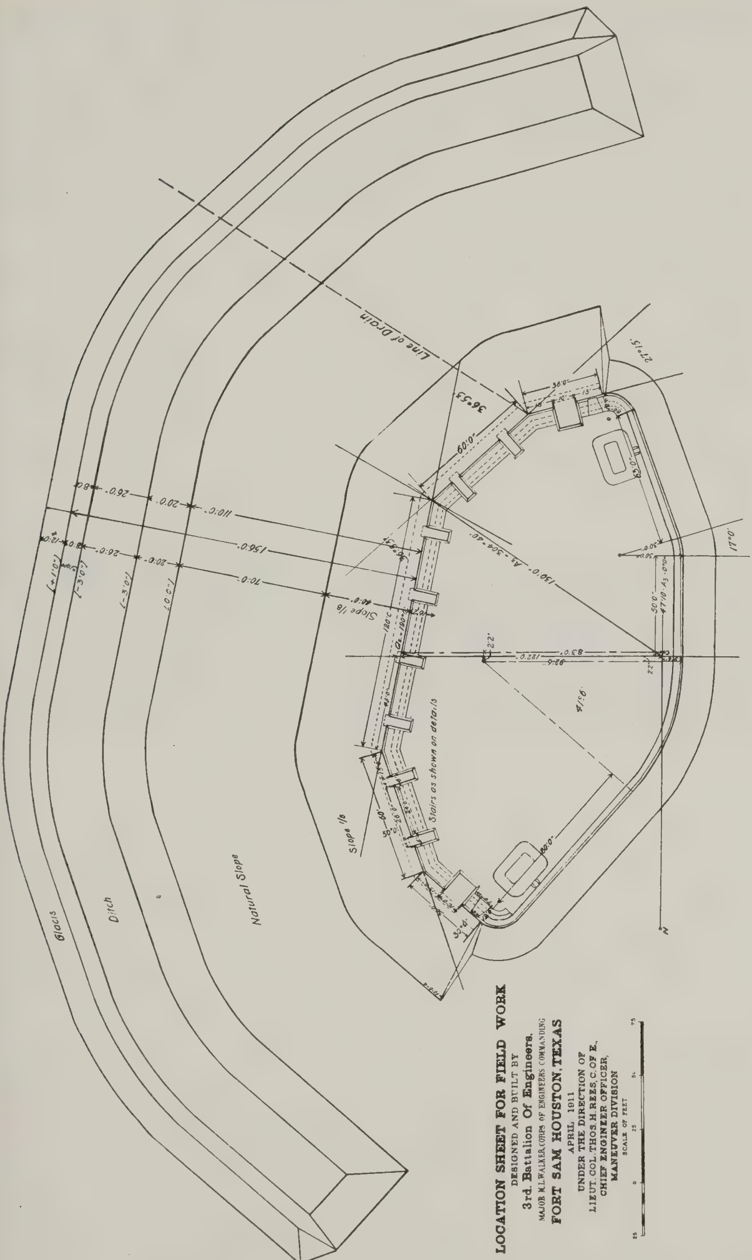

Glacis
Ditch
Natural Slope
Line of Drain
Slope 1/6
Slope 1/3
Stairs as shown on details
LOCATION SHEET FOR FIELD WORK
DESIGNED AND BUILT BY
3rd. Battalion Of Engineers.
MAJOR M.L.WALKER,CORPS OF ENGINEERS COMMANDING
FORT SAM HOUSTON, TEXAS
APRIL 1911
UNDER THE DIRECTION OF
LIEUT. COL. THOS. H. REES, C. OF E.,
CHIEF ENGINEER OFFICER,
MANEUVER DIVISION
SCALE OF FEET

Plate I.

8 yards wide and the remainder 6 yards wide. The entanglement was about $3\frac{1}{2}$ feet high. Posts were set from 18 to 22 inches in the ground and, on account of the hardness of the ground, were very firm with this depth. It was necessary to excavate about 1 foot of the depth, as it was impossible to drive the posts the full depth. They were thoroughly tamped after driving.

Part of the entanglement was built of galvanized wire and part of black painted wire.

The labor involved was as follows: Labor, 413 man-hours; supervision, 83 man-hours.

From above figures, one man can construct 2.85 square yards of entanglement per hour. The figures given in the *Engineer Field Manual* are 3 yards per hour, but it is thought they are intended for smooth wire, which is easier to construct. Forty-five per cent of the above labor was for setting posts, which is a higher percentage than the average.

About 29 feet of wire per square yard was required.

Two infantry companies at war strength were formed from Major Gerhardt's battalion of the Tenth U. S. Infantry, and experiments made to see what length of time was required to get from bombproofs to parapet and vice versa. One hundred and forty-four men were placed in the main trench along the parapet, there being two squads in each 24-foot section between traverses. It was found that the men could fire without inconvenience when placed with this small interval. The time required for the sections to come up from the bombproof and open effective fire to the front varied from ten to fifteen seconds for the different sections. The time required to get under the bombproof cover from the parapet was from eighteen to twenty seconds. The time required to reinforce each of the four end sections from the flank of the gorge trench with two men was forty seconds, time being reckoned from the time the two men at the parapet fell out.

REMARKS.

The work of excavation in the main trench would be greatly facilitated if the traverses were cut out and afterward replaced, thereby permitting the use of plows and scrapers in the trench. This would necessitate the use of sheathing about 3 inches thick, and would be more expensive. Based on the figures for the ditch, the time necessary for the excavation of the main trench would be about two hundred and fifty scraper hours, using drag scrapers.

As conditions for excavation in the trench are not as favorable as in the ditch, the time would probably be somewhat greater than this in spite of the shorter haul. Using twelve scrapers, the time of excavation would be about three days of eight hours each, and two days would probably suffice for the placing of the timbers, revetment, steps, and back fill. Sand bags would be filled during excavation and scrapers would be used for back filling.

If the plow were used to loosen the earth and the earth thrown out with shovels by as many men as could work conveniently, the time would be somewhat greater. Of the 4,163 man-hours required

Fig. 1. View from in front of parapet.

to complete the main trench, about 3,600 were for excavation; the use of the plow would probably decrease this to about 2,000. Assuming 5 men with shovels in each large section and 2 in each lower machine-gun section, the time of excavation would be about 34 hours, making the total time of completion about 6 days instead of 5.

To complete the ditch and entanglement in 5 days would require 2 plows and 22 drag scrapers, equal to 64 plow-hours and 704 scraper-hours, or 4 days for the ditch and 50 men one day for the entanglement.

The ditch does not appear to me to be worth the labor necessary to dig it, as its only value, aside from furnishing earth for the parapet, is to conceal the entanglement. The earth for the parapet could be gotten nearer the parapet.

The entanglement of black wire is much superior to that of galvanized wire on account of greater difficulty in seeing it, it being almost invisible at 50 yards.

BILL OF MATERIALS REQUIRED.

Lumber.

102 pieces 6″×6″×5′6″.	10 pieces 6″×8″×11′.
47 pieces 10″×12″×14′.	140 pieces 6″×8″×12′.
4 pieces 10″×12″×16′.	36 pieces 6″×8″×13′.
126 pieces 2″×12″×12′.	72 pieces 6″×8″×14′.
6 pieces 2″×12″×8′.	50 pieces 4″×4″×8′.
100 pieces 2″×12″×3′6″.	315 pieces 3″×3″×5′ for each 1,000 square yards of entanglement.
40 pieces 6″×8″×6′.	
2 pieces 6″×8″×10′.	

In addition there were used 760 board feet of 2 by 12 inch, and 500 board feet of 2 by 4 inch in variable lengths.

Nails: 200 pounds 60 penny, 200 pounds 40 penny, 200 pounds 20 penny.

Barbed wire: 1,450 pounds per 1,000 square yards of entanglement.

Staples: 40 pounds per 1,000 square yards of entanglement.

Grain bags, 2,500; twine, 12½ pounds.

Figures 1 and 2 show the plans of the work and Plates I and II give the rear and front views after completion.

Comments

Maj. M. L. WALKER*
Corps of Engineers; M. Am. Soc. C. E.

The idea of constructing a semi-permanent fieldwork for the training of engineer troops and to remain as an object lesson for any who chose to visit it, was conceived by Lieut. Col. Thomas H. Rees, Corps of Engineers, and to him belongs the credit for the general design. The details were worked out by Capt. L. V. Frazier, and First Lieut. J. J. Kingman, Corps of Engineers.

The work was assigned to Company K, Engineers, Capt. L. V. Frazier, commanding, and was practically all done by this company, the other companies assisting for a short time with small details.

Blueprints (Plates I and II), are herewith, showing the plan and sections of the work. The idea in its design is that the troops for its defense remain in the bombproofs under the parapet until the fire of hostile artillery ceases. The parapet is then quickly manned in ample time to give approaching infantry a sufficiently warm welcome. The object being to get the maximum volume of

*Commanding the Third Battalion of Engineers during the construction of the redoubt by K Company. Comments accompanying original report.

fire in a minimum of time accounts for no head cover being employed.

The details of the construction of the work are contained in Captain Frazier's report, also enclosed herewith. The actual cost of the timber and wire in this work was $1,000.00. Empty grain sacks for revetment were supplied by the quartermaster of the camp.

As noted in Captain Frazier's report, Maj. Charles Gerhardt, Tenth U. S. Infantry, with his battalion, kindly cooperated in demonstrating the possibilities of the work, with the results set down in Captain Frazier's report. There was ample space for the two companies at war strength in the bombproofs.

The machine-gun platforms were worked out with the guns in

Fig. 2. View from in rear of parapet.

place and served by their regular personnel and were pronounced very satisfactory.

I differ with Captain Frazier as to the value of the ditch and believe it should always be used if time and facilities for its construction are available. It protects the entanglement from artillery fire and introduces a most disconcerting element of surprise for the infantry at the critical period of their advance.

This type of work appears to the undersigned to be a most excellent one and it is suggested that the drawings and such data and photos as are necessary to a clear understanding of it be published in the Engineer MEMOIRS.

Discussion.

Capt. C. O. S. SHERRILL
Corps of Engineers

The two-company redoubt, built by Company K, Engineers, is an excellent illustration of a type of work that will be much used in

future wars, suitable for a small body of troops protecting one of the minor key points of a position. It is to be regretted that the article describing the work did not go into the tactical use which would properly be made of it. In the construction of such a work for instruction purposes, it is very desirable to have not only the details correct, but to have the location of the work determined by certain assumed tactical conditions and by the topography of the position. The work then becomes an object lesson not only in technical design and execution, but also in the tactics of its defense and attack. It is also difficult to estimate correctly the merits of such a work without some assumptions being made as to the rôle it was designed to play. For example, if this work were an element of an extensive line in perfectly flat terrain, it might well be of less depth and have less provision for flank fire. Moreover, the number of machine guns available for flank fire would have a large influence on the trace of the trench.

The trace of this work is an improvement on the first Fort Riley Redoubt, in that it is more nearly a straight line, and thus does not subject one part to enfilade from fire directed against the opposite flank, gives more room inside the trench for a given excavation, and does not have the considerable dead space in front of the salient. It was found in the firing tests at Fort Riley that the angle at the salient should be 150 degrees instead of 120; this work approximates to the recommended trace, and, in addition, has no salient angle to interfere with the fire directly to the front where the maximum volume of fire is usually required.

The lateral communications along the trench would be somewhat restricted, due to the necessity of moving entirely around the work in coming from the rear under cover. This movement would be considerably restricted, too, by the extent of the overhead cover to be passed. The necessity of lateral communications is more or less relieved by the large amount of this overhead cover provided, which allows a larger part of the garrison of the work to be in position during the early approach of the enemy. It would, however, not be desirable to keep more than a trench guard in the bombproofs for periods of more than a few hours; and the necessity of rapidly manning the parapet from a position in rear would indicate better means of accomplishing this end. The omission of a central covered communicating trench leading to the rear was, no doubt, due to the desire to economize as much as possible on labor, in those parts of least instructive value.

The advisability of including a ditch in such a work is open to argument. It requires a considerable amount of extra labor, but furnishes an obstacle in addition to that formed by the wire entanglement which it partly conceals from the enemy. The triangular type of ditch seems to me to be preferable, because scrapers and ploughs can be used in it more readily in throwing up the parapet and the wire entanglement is much better protected from fire and view. The wire entanglement also affords a more formid-

able obstacle, as it extends entirely up to the foot of the counterscarp. By decreasing the height of the posts gradually toward the parapet, the entanglement could be made entirely invisible to the enemy, due to the cover of the glacis and ditch. The element of surprise afforded by an entirely concealed entanglement adds enormously to its value and would throw an enemy into confusion, whereas if its presence were known the advance would be prepared to cross it. The location of the entanglement is not shown on the drawings, but is presumably just in rear of the ditch. The use of barbed wire is important, and especially the black barbed-wire mentioned in the paper.

A question arises as to the necessity of the construction of bombproof cover over the entire front. It requires a great deal more time to complete these bombproofs and the trench together than if the shelter for the main garrison were constructed in rear of the work, and only enough splinter-proof cover provided in the work for the trench guard. It is thought that the bombproofs in the work should be limited to the necessities of the trench guard, because the morale of the garrison would suffer severely under a long bombardment if required to occupy the bombproofs which form the main objective of the artillery fire.

The machine-gun emplacements are well designed and would give the work a strong flank protection in the intervals. The rear emplacement seems to be somewhat exposed to fire from the front of the work, as the traverse protecting the front emplacement does not shelter the one in rear.

Maj. W. D. Connor
Corps of Engineers

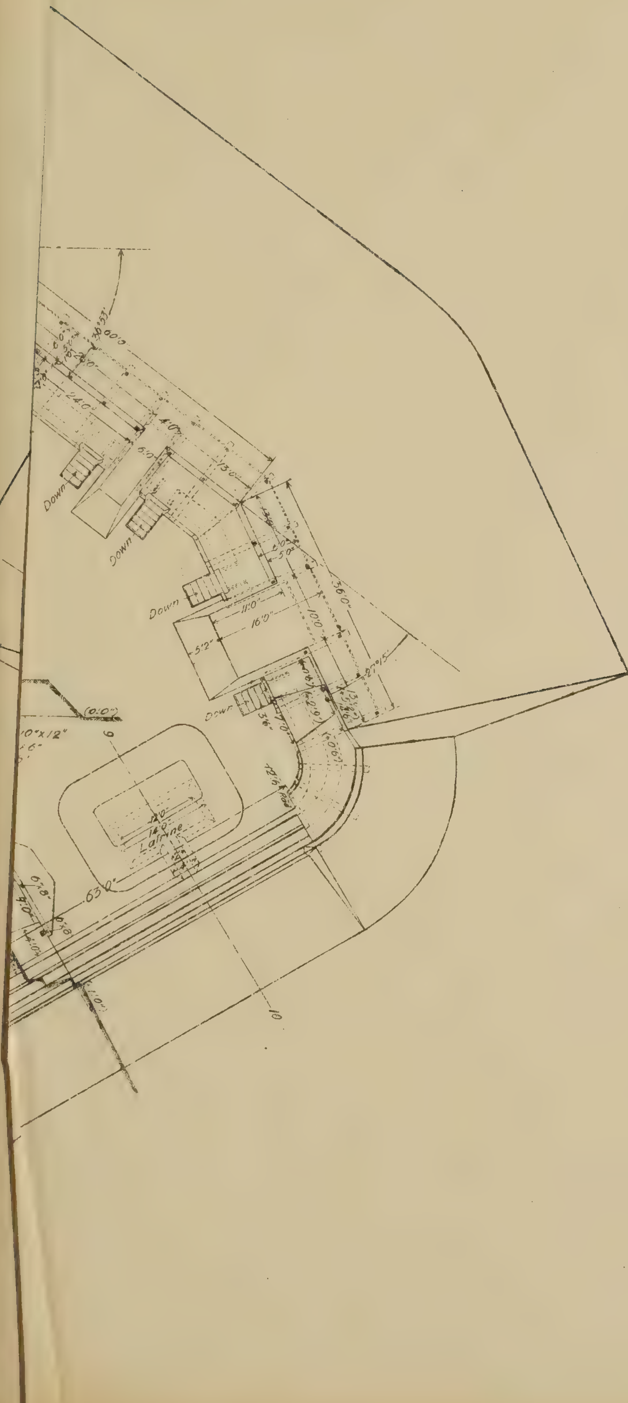
Every piece of experimental work, like the redoubt described by Captain Frazier, is of value and interest; and while agreeing with the general shape of the redoubt as constructed on a level plane, there are certain points wherein I differ and think that the redoubt can be improved upon.

There apparently was no arrangement made for draining the bombproofs and, although such drainage was probably not necessary in Texas at the time that the redoubt was built, an inspection of the drawings will show that the drainage of such deep excavations will often be a difficult matter, and the drainage would be much easier if the depth were not so great. There is apparently no communication between the different parts of the redoubt, except along the line of the exterior trench, and there seems to be no central point of control where the commander of the redoubt can be found and from whence he can readily command all the troops in the redoubt. Adjacent to this commanding officer's station there should be a telephone station and, if practicable, there should be a dressing station where wounded can be attended to without being seen and heard by all the men in the bombproofs.

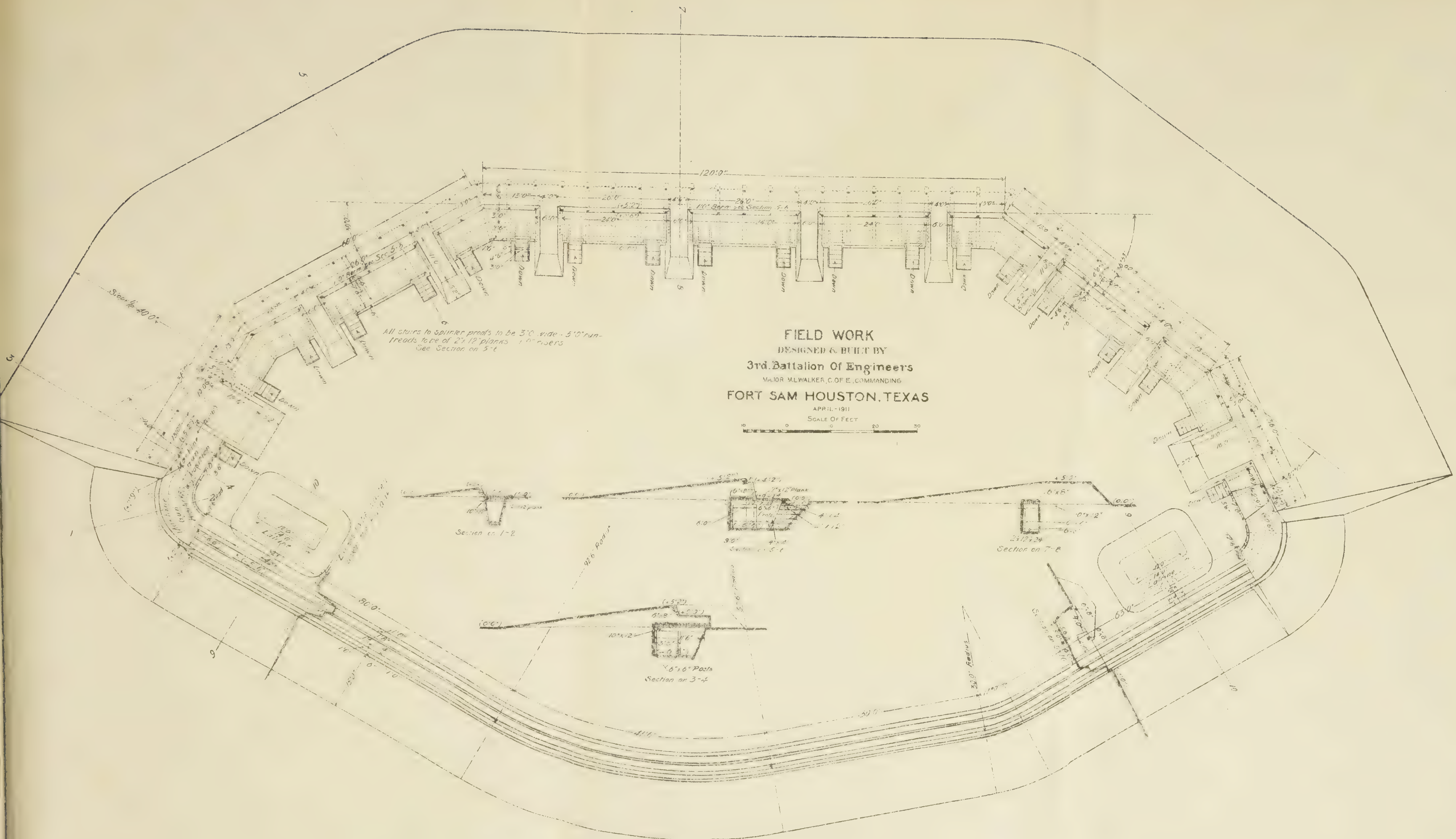
I think that the latrines would be less offensive if placed outside of the redoubt in the neighborhood of the wire entanglements on the flanks. There should be cross-communicating trenches leading from the commanding officer's station to different parts of the redoubt. This would facilitate moving troops from one side of the redoubt to the other in case their presence was needed on one flank or in the rear. The long flat slope to the front, made necessary by such a high parapet and a flat slope, makes a very conspicuous target and one that will be concealed with difficulty. If the height of the crest were 1.5 feet instead of 5 feet, the area of fresh earth exposed would be greatly reduced. During the construction of this redoubt there would seem to be but little cover for the men until the redoubt was completed; whereas, they should have a certain amount of cover at all times, increasing as the amount of work done increases. Placing the bombproof under the parapet gives no opportunity to take advantage of the natural folds of the ground as a part of the cover of the bombproof. In many cases a bombproof can be constructed on the back side of a hill by simply making a shelf in the hillside, while in the redoubt shown in this article the amount of excavation is practically always the same regardless of the lay of the land. In constructing the trench it would seem to be much more economical in labor to use scrapers and animals, cutting out all material at first and then building the traverses of sand bags.

The foregoing are minor differences, but there are two points which I believe are fatally defective in the redoubt constructed. A glance at Fig. 3 shows the rear view of the front trench and the rear closing trench of the redoubt. If a very small party broke through the line between the redoubts and got under cover in rear of one of the redoubts they could make the front line absolutely untenable without approaching within half a mile of the redoubt itself, as there is absolutely no rear protection to the front line, and the men at the parapet are completely exposed to the rear. The second material point is the entanglement, which is shown in Figs. 1 and 2. Apparently there was no intention of making the entanglement closed entirely around the redoubt, and I consider this another vital defect. It is not necessary that the trench line be continuous so long as there are detached parts of the trench facing in all directions, but a closed entanglement is absolutely necessary in order to prevent the work from being carried by a sudden rush.

As a type, I do not think that this should be considered. I think the ditch covering the wire entanglement is a good idea, provided that there is time to construct it. I do not believe that it is essential, however, and believe that such a ditch should be the last part of a work constructed.



Down
Down
Down
Down
Latrine
63'0"
16'0"
11'0"
36'0"



FIELD WORK
DESIGNED & BUILT BY
3rd. Battalion Of Engineers
MAJOR M.L.WALKER, C.OF E., COMMANDING
FORT SAM HOUSTON, TEXAS
APRIL-1911
SCALE OF FEET
10 0 10 20 30
120'0"
All stairs to splinter proofs to be 3'0" wide - 5'0" run - treads to be of 2"x12" planks
See Section on 5-6
Section on 1-2
Section on 5-6
Section on 7-8
Section on 3-4
6"x6" Posts
Down

Lieut. Col. E. R. STUART
United States Army; Professor of Drawing
United States Military Academy

The work appears to conform well to the requirement of protection to the garrison and inconspicuousness as a target, judged not only by the plan and section, but also by the photographs. It is not clear that the barbed-wire entanglement is carried entirely around the work as it should be.

Such a work must be viewed in the light of the function it should perform in a defensive line. Merely as a question of frontal defense, a simple trench can be made strong enough to suffice for that when provided with proper auxiliaries for the protection of the defending troops. The function of a redoubt is not only to provide for the frontal defense in the section of the line that it occupies, but it should also serve to limit any success that the enemy may gain by offering a sufficient defense to the flanks and rear to be able to withstand assault from these directions. Furthermore, the redoubt must possibly withstand such assault when the morale of the defending troops has already been shaken by failure of adjacent portions of the line. It is therefore highly important that under such trying circumstances the garrison should have the material and moral assistance of a well-constructed obstacle, and the barbed-wire entanglement should be carried all the way round the work. The work of constructing a redoubt is largely wasted unless this condition is complied with. Protection against infantry assault unaided by artillery need alone be figured on from the directions of the flanks and rear, as it is not to be assumed that the enemy can support such assaults by artillery if the redoubts or strong all-around works in the line are in proper defensive relation with each other.

Head cover, and particularly bombproof lookouts, should not be omitted in the construction of a redoubt where so much labor is involved. The latter particularly are essential, even in a simple trench, to guarantee thorough observation of the foreground while under fire.

Finally, it is thought that a correct conception of the function of a redoubt in a defensive line is quite as important as the form of the redoubt itself. In my opinion, no redoubt should ever be built, even for instruction, unless it figures as a part of some defensive line, and can be seen in its proper relation to the other elements of such line.

The defensive elements are in themselves simple. Anybody can build a trench or a work if he has a type to go by. The art of fortification lies in the proper location and coördination of the elements of the defensive line. An intrenched company is a unit in such a scheme of fortification, and training in intrenching is an essential element of the training of infantry. So, also, is a redoubt, properly constructed, another element to be assumed. The problem is not the intrenching of a company or the construc-

tion of a redoubt, but the combination of these elements in a line such that it can defy the attempts of the enemy.

Maj. M. L. Walker
Corps of Engineers

Covered observation stations would be indispensable, to be occupied by lookouts for reporting the movements of the enemy. One such station upon each flank should be sufficient.

Since the garrison normally is supposed to remain in the bombproofs until hostile artillery fire practically ceases, it is questionable whether so much valuable firing space upon the main face of the work should be occupied by traverses.

It is not made clear in the drawings, but there was no dead space in the ditch, all portions of it being reached by rifle fire from the parapet.

The troops were not supposed to occupy this work for any great length of time, and hence 12 square feet per man was provided in the bombproofs. A communicating trench, entering the gorge trench near the right flank, led to a well sheltered area to the rear of the work where the living quarters of the garrison were supposed to be.

This work was designed as one of a chain of mutually supporting defenses. The trace is not intended to be typical, but was adapted to a certain hill upon which it was originally intended to locate the redoubt. Just as work was to commence, this hill was needed for camping purposes, and it was then necessary to utilize the only remaining site. This last was a flat and the original plan was adhered to, as there were no features rendering alteration desirable.

Too much weight should not be attached to the time elements deduced from this work. During construction, the temperature was excessively high, permitting only a few hours work each day, and there were many interruptions on account of other duties. It is believed that calculations based upon the results here obtained will give excessive times.

The inconspicuousness of the work is shown by the photograph taken from about one hundred yards in front. A small amount of work for grassing the slopes and providing a background of mesquite bushes would have rendered its location practically impossible, except at very short ranges.

Each interval between traverses—24 feet—was occupied by two squads. In the tests by the battalion of the Tenth Infantry, all sixteen men of these squads were in the firing line. The men stated that they had ample room, and photographs taken at the time show such to be the case. This gives one rifleman for each 18 inches of parapet, a greater massing of men than has heretofore been considered practicable. This result is obtained by the provision of a comfortable and commodious elbow rest, the men taking

position, of their own accord, with their right sides very much refused, thus greatly reducing the space occupied laterally.

The most interesting question in regard to this redoubt is whether head cover should, or should not, be employed. Without it, the enemy can, until the near approach of the attack, prevent by shrapnel fire, the manning of the parapet. In this case the garrison play a passive part until the attack has advanced to, say, 300 yards. With head cover, the garrison can combat the advance at the long ranges. Which is preferable?

It is believed that the question becomes one of the morale of the two sides at the critical time of the advance to the close attack. If the advantage in this respect is with the defense, the attack will be repulsed; and this appears to the undersigned to be the case. During hostile artillery fire the garrison has been safe under cover. It may become somewhat nervous, but it has suffered no casualties. When the artillery fire ceases, immediate relief will be felt, an impression of advantage over the enemy will be experienced, and the garrison will be more than anxious to follow this up by fire from the parapet.

During the distant advance, the enemy has been played upon by the artillery of the defense, so it has not been entirely unhindered. The fact that no fire has come from the work itself will be disquieting rather than otherwise; it introduces an element of mystery. The attacker will be asking "Why is our fire not returned?" and will suspect some sort of ruse. The effect of sudden and heavy fire from the parapet will be more or less that of an ambuscade.

In a footnote on page 92, "Principes et Themes Tactiques," Normand, the following quotation is given from General Hamilton's "Russo-Japanese War."

"A Japanese officer declared in Manchuria: 'When the enemy directs a steady fire upon us it is disagreeable, but nothing is so impressive as the absolute silence of the guns of the defense.' "

Lieut. W. D. A. Anderson
Corps of Engineers

The lines of this work indicate that it is adjusted to the terrain. The traverses furnish ample protection against enfilade, but the gorge trench is not so protected. The large ratio of depth to front is quite noticeable. It would seem better to have the gorge trench run straight across from latrine to latrine, saving length and exposure, and to use the surplus earth to construct traverses.

The bombproofs have an area of 9 by 24 by 9 feet, or 1,944 square feet, excluding the passages under the traverses. According to "Engineer Field Manual" figures, this gives sufficient space for one company to have sleeping quarters or for three companies to take temporary shelter. If the work is to be manned by two companies, bombproof shelter would have to be constructed outside for one of them; or better, if constructed inside as a second line, it

would improve the communications within the work and would furnish a parados for the gorge trench.

The man-hours of labor required show plainly the difference between the theoretical amount of work per man per hour deduced from the work of picked men working a few hours and the actual amount of work that can be counted on from average soldiers under service conditions.

The test shows that sixteen men can fire from a 24-foot length of parapet. It would be of interest to know if the officers who conducted the test consider that the efficiency of fire would be diminished by assigning twelve men to this length of parapet, reducing the rifles 25 per cent, but also reducing the interference and flinching, with consequent gain in individual accuracy and rapidity of fire.

The value of the practical instruction derived from such examples as this field work and the one at Fort Riley can hardly be overestimated. It gives the officer a definite basis from which to work in studying the general subject of fortifications and increases manifold the value of such study. It has seemed to me that a man can not really grasp the practical details of field fortification until he has actually participated in the construction of a complete work of this class.

It might well be made part of the winter course of training in each battalion to require every lieutenant to submit an original design for a field work to fulfill given conditions, to assemble them to discuss the designs and to select the best, and then in each company to assign portions of the work to the noncommissioned officers and require them to figure on the men, tools, supplies, and time needed to construct their parts. A complete work should be completed about once in three years (enlistment period).

Lieut. JOHN J. KINGMAN
Corps of Engineers

A full discussion of this field work by Lieut. Cols. Rees and Kuhn, Major Judson, and others of experience should be of great value to officers of the Corps in furnishing a clearer understanding of the correct strategical or tactical employment of such works as the one here illustrated.

The work is clearly shown by the drawings, and its construction has been quite freely described by Captain Frazier. The distinctive feature of this design is said to be the placing of bombproofs under the parapet, which enables the defenders to remain under cover until the attacking infantry reached such close quarters that his artillery must cease firing upon the work. The defenders then rapidly man the parapet and open with close-range fire upon the attackers before they can pass the obstacle. This would mean allowing the enemy to advance unmolested to within about 200 yards of the work, and would not often be advisable. It is more likely the

parapet would be fully manned and a heavy fire brought upon the attacking troops from the time they first came within effective rifle range, say 600 to 1200 yards. During the artillery duel and until the enemy comes within effective rifle range all the defenders should remain in the bombproofs, which will accommodate both firing line and support. The novel feature of this design would therefore really seem to be in combining the shelter for the support with that for the firing line, instead of holding the support in a separate shelter somewhere nearby as is usually done.

It has been suggested to me by Maj. Lytle Brown that the gorge trench might better be made nearly parallel to and only about 20 or 30 feet in rear of the main trench. This would give a smaller target, make the gorge trench more quickly accessible, and at the same time afford a cross-fire on the immediate rear of the work. It would also put the gorge trench under the shelter of the main parapet.

As stated by Captain Frazier, the building of this field work at San Antonio involved the expenditure of much time, labor, and material. Although in more favorable soil both the time and labor required would be materially lessened, it would still be too much of a task to permit of use except in the more or less deliberate defense of cities, bases, bridges, or passes, and even in such places would not afford protection against siege guns. It is difficult, if not impossible, to give adequate overhead protection against the large caliber howitzer projectiles by the use of earth and timber alone. At Port Arthur it was found that an arched concrete roof from 4 to 5 feet thick with an equal thickness of earth or sand covering was not penetrated by the 11-inch howitzer shells, even when two struck nearly in the same place. It would seem that in modern siege works the use of concrete, preferably reinforced, is almost imperative.

Capt. W. G. CAPLES
Corps of Engineers

This redoubt shows what is probably the most advanced type of construction. A few years ago we were taught to build thick parapets with steep front slopes. These parapets clearly outlined the work and furnished the hostile artillery with a good target. Such a work would now be practically untenable. Since the introduction of rifled guns, the most successful works have uniformly been the least visible. Where there is no natural screen, the easiest mode of concealment is a flat parapet. In the open, a sod-covered parapet can be made quite invisible at 800 yards by making the front slope sufficiently flat. The flat parapet has the further advantage of causing many harmless ricochets, since the shells can not get a good bite for penetration. Just how flat the front slope should be is a matter to be considered in each case. However, in any case, a front slope steeper than 1:10 is rarely admissible and, time and funds permitting, the flatter the slope the better.

Both invisibility and reduction of target are gained by reduction in depth. As works must follow very nearly the natural course of the ground, this results in a more or less crescent-shaped fort. In view of the mode of dispersion of shrapnel, the shallower the fort the better.

I do not favor the gorge trench for works forming part of a closed line. If the frontal attack is strong enough to carry the work, the gorge trench simply secures the victors in their possession of the work. The experiences at Port Arthur seem to show that the line of defense must be a series of strong points connected by a continuous curtain. An enemy that can carry the curtain can probably carry the gorge trench. Unless the gorge trench is supplemented by a caponier to sweep the curtain trenches and the gorge trench itself, I should favor omitting it entirely. For a detached work, however, the gorge trench is quite necessary.

The location of shelters for the garrison in the San Antonio redoubt is a marked advance over shelters some distance in rear of the parapets. The garrison is as safe under the parapet as anywhere, and is where it can man the parapet in the minimum of time.

The San Antonio redoubt is as far in advance of the old style works as is the new 4.7 gun in advance of the old 3.6, and would probably be a harder problem for the 4.7 gun than its predecessors of ten years ago would have been for the 3.6 gun.

Capt. J. A. Woodruff
Corps of Engineers

This type of work appears to me to be a most excellent one for a semi-permanent redoubt. The idea of having the overhead cover under the parapet, so that the troops can man the parapet promptly is especially good.

I do not believe that so much time and labor should be devoted to the construction of a ditch to conceal and protect the wire entanglement. It has been shown by the Fort Riley tests that a great expenditure of artillery ammunition would be required to injure the entanglement seriously. Concealment is a valuable feature, but might be obtained otherwise. (See Professional Memoirs No. 13, pages 103-104.)

The height of the parapet seems unnecessarily great. (See the Shaulautzu and Kangpienhsien redoubts in Major Kuhn's report on the Russo-Japanese War; also Professional Memoirs No. 13, pages 93 and 101.)

The thickness of earth over the bombproofs is greater than required for field guns and not enough for siege or heavy field guns. (See Occasional Papers, Engineer School, No. 39, pages 59, 60, 62.)

There should be better communication between the front trench and the gorge.

The entanglement should extend all around the work. (See Professional Memoirs No. 13, page 95.)

Maj. WM. W. HARTS
Corps of Engineers

After every war of a duration sufficient to test various previously adopted theories, there usually follows a swarm of new suggestions as to proposed changes in weapons, tactics and logistics, out of which must be selected the few best suited to new conditions. The most radical improvement in the means of carrying on land warfare have always been in the rapidity and accuracy of infantry and field artillery fire, but in recent years a phenomenal development has been made in rifles and field guns, and in the power of their projectiles. These changes have brought with them inevitable changes in tactics and these in turn have compelled corresponding changes in many other branches, fortification no less than others.

Not long after the invention of gun powder there was evolved the simple Vauban bastioned trace, well adapted to the weapons of that time, and as these weapons improved, the practice of the art of fortification led step by step to the development of other and more complex systems of defense thought to be better suited to the time. Thence came the systems of Montalembert, Cormontaigne and others. The climax was finally attained in the Noizet front, which was so complicated as to give rise to the pleasantry that the most successful defense possible would be to invite the enemy into its outworks and allow him to become hopelessly lost in its intricate mazes.

Improvement in weapons and advance in the knowledge of tactics forced the abandonment of these types one after the other, and later gave rise to the intrenched camp, involving a chain of detached forts and the outlying infantry positions. This followed the old system in theory, the bastions being replaced by the detached forts, and the infantry defense of the counterscarp, by the outlying trenches of the new system. Under the test of war we have seen these detached forts, which were originally artillery positions, diminish in size and draw nearer, little by little, to the infantry line from their retired location in the rear. But, as field artillery became more mobile and indirect fire became possible, closed works for this arm at length became inadvisable. We now find the detached forts free of artillery and transformed into supporting or pivotal points and finally located in the outermost infantry line of trenches. As a rule, all artillery is now screened by the natural feature of the terrain, is fired from behind hills or folds in the ground, and ought not to be tied down to one spot in closed works.

These supporting points of the present day may be said to vary from the few inconspicuous groups of trenches of the German Field Regulations, on the one hand, to the elaborate Russian redoubts on the other, where little concealment is afforded but, instead, a prominent target congested with troops is presented.

There seems no doubt that intrenched positions can be strengthened against the usual method of attack, when time allows, by

providing more shelter against the enemy's artillery than is afforded by open trenches; this shelter to be so arranged that the defenders may emerge unshaken in good time to meet the enemy's infantry attack after his artillery fire ceases. This requires overhead cover for groups of defenders in close proximity to the defensive line, so located as to enable points on a general line to be occupied quickly and held against a hostile infantry attack until other intervening trenches can be filled with troops for the decisive action. Any simple device that will satisfactorily accomplish this will meet the main demands of a pivotal or supporting point, but it will be seen at a glance that no fixed type or trace can be invariably followed in designing such a field work. On varied ground of considerable relief, only the general principles can be fixed, as the details must conform to the site and it is only on comparatively level ground that our field work will ever approach a type design.

Closed works seem, therefore, to have had their day, except for special localities not exposed to artillery fire and for advanced positions where their advantages outweigh their defects. They are no longer favored by the German, English, or Japanese field regulations for intrenched lines.

Great liberty will probably always be taken by engineers with any type of supporting points in order to secure its full advantages and, after all, this is the best justification for the evolution of a satisfactory pattern.

To test the usefulness of the field work built at San Antonio, it should be considered in the light of the following requirements.

1. It should afford safe shelter against hostile artillery fire.
2. It should enable the immediate development of the full frontal fire of the whole command when required.
3. It should cover its own flanks by strong rifle or machine gun fire, usually for the support of the next similar work, and it should afford a cross-fire in front of the neighboring intervening trenches.
4. It should be capable of defense against reverse attack in case the interval between pivotal positions is pierced, and have wire entanglements or other obstacles entirely surrounding it for protection against night attack.
5. It should present a minimum target to the enemy and be of low command to favor concealment and must have flat slopes for deflection of projectiles.
6. Overhead cover for the entire garrison should be provided, having frequent traverses to localize shell effect, and good interior and exterior communications, protected or concealed.
7. It should be as shallow as practicable in the direction of the line of the enemy's fire to minimize shrapnel effect and have nothing prominent in rear to act as a "shell trap" for over-shots, that is to say, be practically "open" to the rear.
8. Head cover should be provided, preferably by sand bags in

the trench not to be placed until the infantry attack commences, in order not to disclose its position to the enemy's artillery fire.

9. It should be adaptable to the average site, be simple in plan, and involve a minimum of labor and expense.

The San Antonio work meets these requirements far better than most designs recently advocated, and a study of its arrangement will be profitable to every military engineer who may have this sort of work to design or build. The utilization of the bombproof cover as an infantry parapet and the resulting nearness of the garrisons to its firing position seems specially commendable.

The statement of the time and labor required and tools and supplies necessary furnish the great part of the specifications for its construction. These would be very valuable in case the work had to be built under the hurried and excited conditions of an impending hostile movement.

It is noticed that the rate of work during construction fell far below the usual text-book standards. English manuals and our own give 20 cubic feet per hour as a reasonable task for each soldier in soft earth and 10 cubic feet in hard material is given in our Engineer Manual as a fair estimate.

The main trench required 735 cubic yards excavation and took 4,163 man-hours, corresponding to a rate of less than 5 cubic feet per man per hour. The gorge contained 192 cubic yards and required 967 man-hours, or something over 5 cubic feet per man per hour; latrines, 37.7 cubic yards and 332 man-hours, or about 3 cubic feet per man per hour. If the rate heretofore assumed as reasonable is not wrong, it would be desirable to have the reasons for this discrepancy more fully explained.

Maj. C. A. F. Flagler
Corps of Engineers

The redoubt as a field work to be erected by the troops is likely to be a rarity in the future. In permanent works erected in time of peace, or in provisional fortifications at strategic points in the theater of war, erected not in the immediate vicinity of the enemy and generally by civilian labor, the redoubt still has a place. The redoubt at Fort Sam Houston would be an excellent type for provisional works on lines of communications; bridge-heads to check raiding parties and other points where attack in force, particularly by bodies provided with heavy field artillery, is not to be expected.

For "supporting points," "fortified pivots," or whatever term is applied, in a defensive position taken by a field army and strengthened, it is preferable to fortify at intervals the strongest points offered by the terrain with a carefully devised system of trenches, obstacles, bombproofs, etc., adapted to each individual site. Such fortified points will almost never present a symmetrical work. There is a strongly marked tendency in this country for

engineer troops of both the Army and the organized militia to leave a redoubt as a monument of engineer occupation at each maneuver ground or field encampment. The construction of these is, doubtless, excellent for instruction of the troops in trench, bombproof, and obstacle construction, and probably has a beneficial effect on the lay mind of the local inhabitant until the elements have reduced them to ruins. Many such redoubts are scattered over the country. I will plead guilty to one.

It would appear to me much better in these practice constructions of field works, to mark out a defensive line extending across country, select one of the suitable points on this line for a supporting point, assume a fixed garrison for it, and then make a careful study of the ground and lay out a series of trenches so as to utilize the fire of the garrison to the best advantage. The obstacle, bombproof cover, latrines, communications and drainage can then be placed to conform to these trenches. If such a work is planned and executed by engineer troops, they will obtain fully as much experience in working details as in the construction of a model redoubt, and both officers and men will obtain valuable knowledge in the type of the work that is likely to be required in actual war. Problems of this nature have been solved at the service schools on maps, and, occasionally, on the ground, but I know of no case of actual construction.

One vital objection to such construction for practice is the fact that no suitable military name has yet been applied to these works. To build a redoubt sounds military and formidable, to build a "supporting point" sounds like nothing at all. I am convinced that lack of a suitable name has acted as a deterrent in this matter, and one should be adopted as soon as possible.

Some obsolete fortification term applied to works of allied use might be adopted, such as "cavalier," "place-of-arms," etc. An entirely new one would be preferable, such as "mitral," or work where the line is mitered; "anglon" (similar derivation), "crozier" (no derivation whatever). The Seventeenth and Eighteenth centuries showed no paucity of imagination in the invention of names for new military works, and it is to be regretted if the Twentieth develops reluctance thereto. Any name adopted will be a convenience for discussion, and will soon by use acquire the desired military ring.

The Fort Sam Houston redoubt has many excellent features. The elimination of the central traverse, common to so many modern redoubts, is, I think, an improvement. The trench and the latrine profiles appear excellent for service and economy of labor. The arrangement of bombproofs is the best I have ever seen for rapid manning of the parapets, as was shown in the tests made. The work is evidently designed to afford protection only against light field artillery. The 4.7-inch rifle and the 6-inch howitzer would soon render it untenable. The absence of shrapnel cover along the parapet would cripple the defense in the early stages of an

assault, but could be easily added in construction or improvised at a later stage. I agree with Maj. Walker about the ditch. I believe it is almost essential for the protection and concealment of the entanglement.

I have enjoyed the pleasure of satisfactory service with the author on military duty, and I know that whatever he does is well and thoroughly done.

Capt. AMOS A. FRIES*
Corps of Engineers

I have read and studied this article and the drawings accompanying it with a great deal of interest, for the subject appeals to me as one about which we need to arrive at some more definite conclusion than at present. I have also had the advantage of reading all of the comments made. So far as the general trace of the work is concerned, it seems as good as most traces, but the details do not seem to me at all suitable. The bombproofs have at the most 5 feet 2 inches of cover, and this notwithstanding that the security of the firing trench depends upon the durability of the bombproof itself. The firing tests carried out at Fort Leavenworth two and a half to three years ago showed that this amount of cover was entirely insufficient against 6-inch howitzers and very likely 4.7-inch howitzers and, if this is proposed as a semi-permanent work in the defensive line, we must assume that it will be subject to attack by such guns. The very construction makes it doubly poor for the reason that a 6-inch shell will not only destroy a section of the bombproof but the parapet above it, and make the repair of that parapet vastly more difficult in the limited time that might be necessary to repel an attack.

Again, the idea that the troops may remain in these bombproofs until the attackers are within 300 yards seems to me entirely wrong. If those attackers can arrive within 300 yards without effective infantry fire against them they can remain in that position while the artillery keeps the defenders out of sight until they have a sufficient force to maintain a superiority of infantry fire and thereby render the attack much more likely to succeed than if they were forced to be under a strong infantry fire from the limit of the effective range of such fire. If the work is not intended to be a semi-permanent one and thereby subject to the attack of 4.7-inch and 6-inch, or even heavier projectiles, there is no place for it, because it is too expensive in time, labor, and materials for a field work. There would not be time to build such a work in the face of the enemy when the battle was eminent or in progress, and any attempt would be sure to result in failure. A much better arrangement would be the simple standing trench with splinter-proof overhead cover, about 18 inches thick, forming a standing trench equally as good as the one under discussion, besides being

*Director of Military Engineering, United States Engineer School.

one that could be widened to make a passageway in the rear even during battle, and while the parapet was being strongly manned. I believe it absolutely essential that the parapet shall be manned whenever the enemy is within 800 or 1,000 yards. Contrary to the idea expressed in one of the comments, I believe that the effort to keep the men in the bombproofs where they would be subject to all the horrors and dangers of exploding shells, suffocation and suspense, would demoralize them far worse than even greater casualties while occupying the trenches. I am therefore strongly in favor of the trench thoroughly fitted with loopholes of the most substantial character, going to the extent even of providing steel shields for protection against rifle bullets such as were used in the later days of the defense of 203-Meter Hill against the close attack of the Japanese.

I do not think that too much emphasis can be laid on the point brought out by Professor Stuart, Major Flagler and Major Walker, and touched upon by others, that the construction of any redoubt in maneuver camps or elsewhere should conform to the actual tactical location of a redoubt designed to defend a position back of the point where it is built; and, furthermore, that small sections of a redoubt or of two or three redoubts together with the connecting lines of trenches showing the defensive line, is of vastly more value, not only to engineer troops, but to all other troops than is the complete construction of a single redoubt by itself, whether built in a good tactical position or not. Of course, I realize that such positions can not at times be found, having had such a case during one tour of duty with national guard troops, but in that event it is vitally necessary that every effort be made to point out to all troops observing the redoubt, its deficiencies as regards its tactical location.

I also heartily agree with the view taken by several that a wire entanglement should completely surround the work and should, where time will allow, be protected by a parapet in its front so arranged that there will be no dead space either in the wire entanglement or in front of it, and also that there should be a strong gorge trench capable of affording a powerful and effective fire to the rear and some means of protecting the defenders in the front trench from reverse fire which, as Major Connor points out, is wholly lacking in the redoubt under discussion.

In this connection I do not agree at all with the contention of some that the gorge trench is of no value for the reason that if the enemy breaks through the line of trenches between the redoubts the redoubt itself must fall and the position be captured. I am aware that one writer gives the impression that had the Japanese been able to break through the curtain wall between the forts on Wantai Hill that not only the forts would have been taken but that probably Port Arthur would have fallen in that particular assault. This conclusion is open to question, and especially in view of the fact that during the two weeks' continuous attacks on 203-Meter Hill the Japanese succeeded on several occasions in

breaking through on the flanks and getting into the edge of the fort itself and yet were just as often driven out until the final capture of the place after its nearly complete demolition by heavy mortar fire.

To sum up, my objections to this redoubt are, first, by putting the firing trench immediately over the bombproof you subject both to ruin by the same shell and make the repair of the firing trench much more difficult; second, the bombproof cover is too light for a semi-permanent or permanent work; third, the redoubt is too elaborate for a field work that must be thrown up in the presence of the enemy or when the enemy may be expected at any hour; fourth, the absence of loopholes makes it a very inferior firing trench for any stage of the attack when the besiegers are in rifle range, and fifth, the relief is such that the defenders along the front of the work have absolutely no protection from fire from rear and are thereby subject to being easily driven out by any small force of the enemy that may break through the trenches between redoubts, a condition rendered doubly dangerous if the wire entanglement does not entirely surround the work.

Mr. Ernst Kuhl*

Civil and Mechanical Engineer, United States Engineer Department

Ernst Kuhl was born at Saarlouis, Germany, February 9, 1843. At the age of 18 he was graduated from the gymnasium in Treves at the head of a class of which he was the youngest member. In 1865 he completed the course in civil engineering at the Polytechnicum in Carlsruhe, Germany. For several years after graduation he was manager of a smelting works on the river Rhine, and for a time filled the position of engineer on a North German Lloyd steamer.

In 1872 he came to the United States and settled in Buffalo, N. Y., where, early in 1873, he entered the office of the Tenth Light-House District. Here he was employed as engineer and draftsman for three and a half years under Lieut. Col. Chas. E. Blunt, by whom he was very highly esteemed.

From 1876 to 1878 he was in business for himself, manufacturing special apparatus for the transmission of power.

In 1878 he reentered the Government service, where he remained until his death. From 1878 to 1900 he was employed as mechanical engineer and draftsman on the improvement of the Missouri River—under Col. Chas. R. Suter from 1878 to 1896, and under Lieut. Col. Amos Stickney from 1896 to 1900. Capt. Theo. A. Bingham and Capt. J. C. Sanford were secretaries of the Missouri River Commission during a portion of this period.

All these officers have written of Mr. Kuhl in the highest terms.

His services were found particularly valuable in designing the machinery for the towboats *Arethusa* and *Atalanta* and the snagboats *Wright* and *Suter* (now in service as the *Missouri*).

From 1900 to 1910 he was superintendent of the de-

*By J. W. Woerman, Assistant Engineer, Western Division.

partment of mechanical engineering of the United States Engineer Office at Pittsburg, under Maj. Wm. L. Sibert and Lieut. Col. H. C. Newcomer. Here he designed a number of steamboats and dredges and the structural steel and operating machinery for a number of locks and dams on the Monongahela River.

Under date of January 26, 1912, Colonel Newcomer wrote: "His long service and professional skill made him a most valuable assistant. He was a man of the highest integrity, of fine technical ability and sound judgment, industrious and loyal."

In 1910, following the death of his wife, to whom he was married in 1873, he was transferred to the United States Engineer Office at St. Louis, where he resided with his younger daughter, Mrs. Martha Kliefoth. Grief for his wife rapidly undermined his constitution, and he died December 29, 1911, at Minneapolis, Minn., where he was visiting his elder daughter, Mrs. Helen Zahn.

On account of his quiet and retiring ways, his circle of friends was not as large as it might otherwise have been, but those who knew him recognized in him a man of sterling qualities and high ideals. His disposition was sweet and lovable; his character noble and honorable. His death is a keen loss, not only to his relatives and friends but to his adopted country, which he served so faithfully for a period of thirty-four years.

Book Reviews

Engineering as a Vocation. *By* Ernest McCullough, C. E. Published in 1911 by David Williams Company, 239 West 39th St., New York, N. Y. Price, delivered, $1.00.

This is a most timely book and one for which there is a very great need. It should be read by every engineer, every employer of engineers, every professor and every student or prospective student in engineering schools, and by fathers and mothers who contemplate having their sons study engineering.

The book is valuable to the engineering profession at large because it holds out no rosy views of high position and large salaries to every man regardless of ability, energy, or general fitness, who may get a degree as an engineer whether civil, mechanical, electrical, mining, or other title that some school may adopt.

The first chapter deals with the definition of engineering, and with the differences between various engineering schools in the United States and abroad. The author quotes two definitions of engineering which are well worth studying: first, by Thomas Tredgold, "The art of directing the great sources of power in nature for the use and convenience of man;" and second, Wellington, "Engineering * * * is the art of doing that well with one dollar which any bungler can do with two after a fashion."

In this day of hasty, unthinking specialization, one appreciates finding an author broad enough to take the stand that the man who specializes narrowly as a student is not only very apt to fail to become a great specialist in later years, but is far less apt to become reasonably successful than is the man whose general education is broad and who specializes, if at all, only in the very last part of his student course.

Chapter IV is particularly valuable to the man who must rely on home study, as he not only discusses the general lines of study, but gives books, publishers and prices. In other words, he gives something definite instead of only glittering generalities, as is too often done.

Chapter V is given over to a discussion of how to hunt and hold a job. All engineers need to study this subject, and especially the young engineer, not only for his own good but for the good of the profession in general. Many a good engineer has failed to get a good position simply through lack of knowledge or training of how to go after such a position.

Chapter VI is given to a discussion of whether it pays to study engineering. This discussion is on the same broad liberal lines

characteristic of the entire book, and as the author rightfully puts it, whether or not engineering pays depends upon what the one who asks the question means by "pays." If he means the piling up of great wealth, it won't pay. If he means following a vocation that leaves evidence of work well done and which helps the progress of the human race as a part of the compensation, then the work of the engineer certainly pays.

The book ends with a couple of editorials from two engineering publications on The Over-crowding of the Engineering Profession, and What Should be Done to Raise the Standards in the Profession and Obtain for Engineers the Compensation to Which Their Work Justly Entitles Them.—A. A. F.

Dredges and Dredging. *By* Charles Prelini, Professor of Civil Engineering, Manhattan College, New York. Published by D. Van Nostrand Company, New York, N. Y. Price, $3.00 net.

This book covers a field on which very little has heretofore been written except in papers published here and there by engineering societies and engineering publications. It covers the subject of dredging in its various forms, including, as is proper, subaqueous rock excavation, dredging for gold and other precious minerals, and dry-land dredging in general. It is a book very much needed by the student of dredging, by the engineer who has to plan and carry out dredging operations, and by the contractor or other person who actually does the work or furnishes the money for that purpose.

Heretofore in dredging, as in many other phases of development in America, there has been so much work to be done and prices generally so good that close attention to obtaining the proper machinery for the work was of far less importance than getting anything that would do the work; but to-day this condition has largely passed, and the engineer or contractor who desires to make a success of a piece of work must study his subject so as to get the best machinery and the best methods for carrying it out.

Considerable space is devoted in the early part of the work to a discussion of ladder dredges, a form of dredge very little known in this country and, as the author says, certainly not as yet appreciated for its true worth. The author has wisely incorporated where possible costs of dredging in its various forms, while at the same time calling attention to the fact that in considering costs one must consider not the contractor's receipts, but the different elements of the cost in order to be able to apply the information obtained from one piece of work to that in a different locality where there may be different wage standards, different prices of materials, etc.

The work is generally excellent, though parts of it are brought up to only about 1905 and 1906. This being the first edition there are a few slight typographical errors, and the half-tone cuts are too often not a credit to the publication through using too coarse a

screen. Nevertheless, these do not detract from the general excellence of the book for all persons desiring to make a general study of the subject of dredges and dredging.—A. A. F.

Power House Design. *By* John F. C. Snell. Longmans, Green & Co., 1911. Price, 21 s., net.

The author has treated his subject largely from the English point of view of standard practice, but broadly enough so that his book is highly valuable in the library of any American engineer, general or specialized.

The treatment is comprehensive, but not drawn out. The plan is logical, and the gradual and logical proceeding from subject to subject is refreshing. The book is profusely illustrated with examples of current practice and with performance diagrams; and tables and formulæ not always found in works of this kind are ready to one's hand.

The author begins with an outline of his work and then proceeds to treat in detail the great sub-divisions as regards steam-power plants, these are sites, buildings, boiler plants and auxiliaries, steam engines and auxiliaries, and condensing plant. There is an excellent discussion of the modern forms of turbines—mixed pressure and exhaust steam types.

Power plants using other sources of power than steam in their prime movers are then taken up and discussed in detail, but only in so far as they are inherently different from steam plants. The internal combustion engine plant is placed before the student in clear and concise form, but the author confines himself almost entirely to foreign types, a restriction not found in other portions of the work. Hydroelectric plants for various heads are taken up, particularly the Niagara Falls plant of the Ontario Power Co.

The author in another division discusses the design, construction, and materials of switchboards and appliances; the practical points of generators and transformers for converting the energy of the prime movers into electricity; and the uses of motors in driving power plant auxiliaries.

The book ends with a chapter on small isolated power plants, and substations of large plants.

Throughout the book the reader is impressed with the logical plan, the clarity of expression, the comprehensiveness of the information and its concise form of presentation. As mentioned, the volume is not bulky and yet contains information which can usually be found only by consulting several works.

It would appear to be the most satisfactory single book of this nature for the general engineer which has as yet been published.

—W. F. E.

FROM ROUGH RIDER TO PRESIDENT. A translation from the German of Dr. Max Kullnick by Frederick Von Reithdorf, Professor of Modern Languages, Monmouth College, Illinois. Published by A. C. McClurg and Company, Chicago, Ill. Price, $1.50 net.

A most excellent book and very well written. It is presumed that the original German was excellent, but however excellent it may have been the translator has done it full credit. The title indicates that the book covers Ex-President Roosevelt's life only from the time of the Spanish War up to the date when it was written in 1904. This is by no means the case, as the author devotes fully one-half of the book to his life from the time of his birth to the breaking out of the Spanish War, and this in many ways is one of the most valuable parts of the book and should be read by every boy and young man. It shows, first, how with proper care and determination a boy with a comparatively weak constitution may develop into a strong, vigorous manhood.

The author is evidently a great admirer of outdoor life, as is Mr. Roosevelt himself, who believes it makes for vigor, endurance and independence. One of the valuable lessons in the book is that a vigorous stand for what a man thinks is right, no matter what the occasion, will win out if the man simply has the nerve to stay with it in the face of preliminary defeats. The whole book is an inspiration to high ideals and a hatred for anything that is unjust or unfair.

As one reads the book the feeling gradually comes to him that it could not have been better written by an American, and that unfortunately the number of Americans who have as accurate a knowledge of American history as has the author of the book is entirely too small.

We can heartily commend this book as one that should be in the library of every boy between the ages of 12 and 60.—A. A. F.

PROGRESS AND PROSPERITY. *By* William De Hertburn Washington. Published by the National Educational Publishing Company. 1911. Price, $4.50.

If the entire cost of gathering the pictures, facts and fancies, and writing and publishing them in the above-named book were not paid for by the railroads of the country, then the latter are the beneficiaries of a philanthropy unbelievable in this age of business idolatry.

The book is gotten up in readable shape, and by introducing the various phases of the subject discussed with pictures of Indian massacres, real and fancied, pictures of first railroads, or conditions before there were any railroads, the readers' interest is obtained.

But, whether it be banking or hog-raising that the author is writing about, the ending of the chapter is the same—some parallel, simile, or less subtle way of trying to make the American public

believe the railroads of the country deserve better treatment and more revenues than they are now getting.

That the railroads have been the greatest factor in the marvelous growth of the world in general and the United States in particular in the last seventy years no one will deny; but, while the benefit to the general public has been enormous, it is small compared to the harvest reaped by the Wall Street operators who control the roads to-day.

The book is full of parallels like this:

"Were the same freight charges that were paid a hundred years ago levied to-day, the Rail Roads of the country would get more money for carrying our present volume of mail alone than the total of their present receipts for carrying passengers, freight, mail and express."

Does the author think the American public made up of children? It would be just as sensible to compare the cost of printing "Progress and Prosperity" on the stones and bricks of the ancient Babylonians with the cost of printing it on the Hoe presses of to-day.—A. A. F.

Selected Articles of Engineering Interest

Compiled by Henry E. Haferkorn, Librarian, Engineer School.

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering-Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
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WIRE ROPE.

Uses, care and strength of wire rope. (39), Jan. 15, 1912. D. I.; Feb. 1, 1912.

Editorial Notes.

Trench Digging by Dynamite.

Dynamite has been used to a considerable extent during the past few years for ditch-digging in drainage work, and photographs of the results led to the belief that the same method could be applied with advantage to the construction of trenches for fortification work. This method would hardly apply to trenches or other elements of land defense constructed in the presence of the enemy, but there would seem to be many opportunities for its use in connection with the land defense of cities where the work would be done largely by hired labor, or in the construction of fortified positions in the rear of any army where few men would be available for the work, and where considerable fortification work would be necessary.

The method of procedure is to drive or drill holes, about 2 inches in diameter, into the ground at intervals of from 3 to 4 feet along the desired trench line. A stick of dynamite, more or less, the amount depending upon the character of the ground, exploded at the bottom of the hole will not only blow out a considerable amount of the material, but will loosen the remainder so that the remaining pick and shovel work will be very greatly facilitated. The charge placed in the hole should be such as to produce a common, or "two-line," mine.

In ordinary ground these holes can be made most easily by the use of a solid iron pin, about 2 inches in diameter, sharpened to a blunt point at one end and provided with a ring at the other end. By driving the pin with a maul and withdrawing it by means of a lever, the holes can be quickly and easily prepared. The depth of the hole should correspond to the desired depth of the trench, as the dynamite has very little effect below the bottom of the hole. In very hard material some form of earth auger may be necessary, but, ordinarily, the pin makes holes much more rapidly than does the auger.

This method of excavation has other applications than those for military uses, as it provides an especially cheap and rapid method of digging trenches in wet or marshy ground. In this case, if all charges are under the surface of the water, only one charge need

be fired with a fuse or cap, as all others will be detonated sympathetically, thereby saving a great deal of time that otherwise would be spent connecting each cartridge to the battery.

This method of trench digging was used during the past summer at Fort Foote, Maryland, by the troops of the First Battalion of Engineers, and the results were so generally promising that further experiments will be made in the immediate future. The same method has been applied to trench digging in the swamp lands of the Mississippi bottoms, and photographs showing the results obtained indicate that a very satisfactory and uniform ditch can be obtained by this method. Rack-a-rock was the explosive used at Ft. Foote, and dynamite that used in the Mississippi bottoms.

Two Decisions Rendered in the Court of Claims of the United States, December 4, 1911

These two decisions are on claims for damages alleged to have been caused by the construction of levees along the Mississippi River. The claims are based on the provision of the Constitution that private property shall not be taken for public use without just compensation.

The first case (No. 18274), known as the "Jackson" case, is one in which Mattie W. Jackson et al. entered a claim against the United States for extensive injuries to their property caused by the construction of the general levee system, and is typical of a large number of claims. It is based on the fact that the construction of the levees has so raised the high-water levels of floods in the Mississippi that lands which were formerly overflowed at comparatively rare intervals are now overflowed at much more frequent intervals, and, further, that each overflow continues for a longer time. These lands lie on the east side of the river between the channel of the river and the low range of hills situated from 2 to 6 miles back of the river. No levees have been built on the east side, as the hills protect all but a limited area in which those lands lie. Judgment was rendered for the United States and the case dismissed.

The other case (No. 30471) is one in which George F. Archer and Kate C. Archer entered a claim against the United States for the actual taking or total destruction of 3,696 acres of land out of a total of 6,000 acres in what is known as the "Point Chicot Plantation." This case is quite different from the preceding one and, unlike it, judgment was rendered in favor of the plaintiff.

While the former, or "Jackson," case has to do with the injury to lands on the river side of the levees, caused by the increased height and longer duration of floods due to confining the river between the levee systems instead of allowing it to spread out as in its natural condition; the latter case is one where a dike was built from the general levee system, not only up to the claimants' land, but over that land for a distance of 2,500 feet without the consent of the owners. The claim was based on the fact that this dike, which was built to protect the main levee, not only destroyed the 31 acres of land occupied by the levee and the spoil banks from which it was built, but the remainder of the 3,696 acres through directing the flow of the Mississippi River during floods across it. The flood as thus directed either washed off the good soil or deposited large quantities of sand and gravel, or both, thereby ruining the land for plantation purposes. The court held that this was an actual taking of the land and therefore in violation of the Fifth Amendment to the Constitution of the United States.

The "Jackson" case will undoubtedly be appealed to the Supreme Court of the United States and possibly the "Archer" case also.

There is a good deal of interesting reading in the findings of fact in these cases concerning the condition of the Great Mississippi Basin before the beginning of the construction of levees and at various times since up to the present date. The findings of fact and opinion in the "Jackson" case cover 28 pages, each a little larger than those of the PROFESSIONAL MEMOIRS, and the "Archer" case covers 12 pages of the same size.

The opinions in full in both these cases will be published in the next number of the MEMOIRS.—A. A. F.

Influence of Rifle and Revolver on Needle of Sketching Case.

(Extract from Report of Captain Fries, November 3, 1910.)

(q) The question having come up as to whether or not the rifle and revolver carried by the sketcher would seriously affect azimuths taken with the needle I had the matter investigated. The sketcher wore the revolver in its proper place and had the gun slung across his back in the usual way. Angles were taken so as to show the minimum effect on north and south lines and the maximum on east and west lines, together with a number of intermediate points. A prismatic compass was used because of its sensitiveness, and close reading properties. The result shows that the maximum deviation is less than 1½ degrees, an entirely negligible quantity in road or position sketching.

Concerning Book Reviews.

A correspondent writes us questioning the advisability of including book reviews and selected articles of engineering interest in the numbered pages of the PROFESSIONAL MEMOIRS on the ground that these are not of sufficient interest to be incorporated in bound volumes, and therefore should be unpaged in order that they may be torn out when the volume is to be bound.

We are glad to get such expressions of opinions from our subscribers on this subject or any other connected with the MEMOIRS. In this particular case others than our correspondent feel that the selected articles of engineering interest are of sufficient importance to be included in a bound volume, and, so far as book reviews are concerned, we hope to make them of such value that most subscribers will desire to keep them.

Many publications carry book reviews as a side issue simply for the commissions to be obtained from the sale of books, and as a result the reviews are mainly palaver with that end in view. We are not agents for any books and are not contemplating becoming so. Our only desire is to get hold of the latest books, and by reviewing them place before our readers enough information to enable them to decide whether or not they care to purchase any particular book.

With that object in view we expect to have books reviewed only by persons competent to pass judgment on the subject covered by the book. If the book is good we shall say so, and try to tell in what way it is good. If it is not considered good we shall say so, or else not review the book.

Errata in No. 12

In the last part of the second term of the equation on page 591, the denominator should read $1+\varphi$ and the parenthesis around the numerator should be omitted.

Page 596, line 15, the word "footing" should read "footings," and in line 17, the words "center of" should be stricken out.

Reprinted

The edition of No. 1 of the PROFESSIONAL MEMOIRS became exhausted over one year ago and the number of requests for this number having warranted it, it is now being republished and will be ready for sale by March 15. The price will be the same as the original subscription price, $1.25.

Memoirs for 1912

It is with pleasure that the Editors of the PROFESSIONAL MEMOIRS can announce a 10 per cent net increase in subscribers since No. 13 was issued.

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Corps of Engineers, United States Army, and Engineer Department at Large

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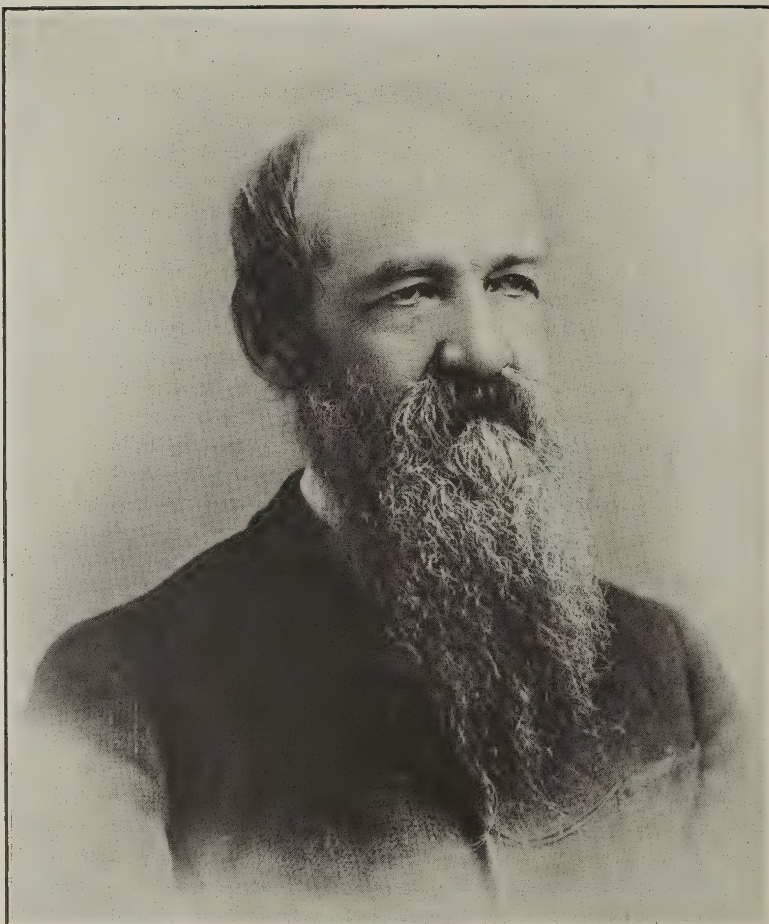


BRIG. GEN. JAMES C. DUANE

CHIEF OF ENGINEERS, UNITED STATES ARMY

1886-1888

BORN 1824—DIED 1897

Power Development at the Falls of the Ohio Louisville, Kentucky

BY

Maj. LYTLE BROWN*
Corps of Engineers

There is one, and perhaps only one, point on the Ohio River where the development of water power is a commercial possibility. The plans for the improvement of the stream for purposes of navigation by the General Government are now well defined and settled for an indefinite future period. As these plans are for the canalization of the stream from source to mouth it might be expected that at each of the proposed dams on the river water power in rather large quantities would be available for use. At each of the dams, save the one at Louisville, the entire head available is created by the dam, and the average lift of these dams is only about 7 feet and reaches 9 feet at a maximum. Furthermore, all of the dams are movable and are intended to be thrown when the natural stage of the river will give a least depth of 9 feet on the obstructing bars. It can be seen at a glance that such conditions are so adverse to the development of water power on a large scale that it is scarcely necessary to go into the question further. But for the development at each site of the small amount of power, 100 horsepower, necessary to operate the works of navigation, which is as a rule required only when the dams are raised, the conditions are such that the authorities have decided to install at each site a small water power plant for furnishing the compressed air necessary to operate the valves of the lock and the gate engines, for the raising of the bear-trap sections of the dams, and for various other small incidental purposes.

At the Falls of the Ohio the conditions are exceptional. A ledge of limestone rock runs across the bed of the stream, making, in effect, a natural dam which at low water holds the upper pool at an elevation of 27 feet above the surface of the lower pool.

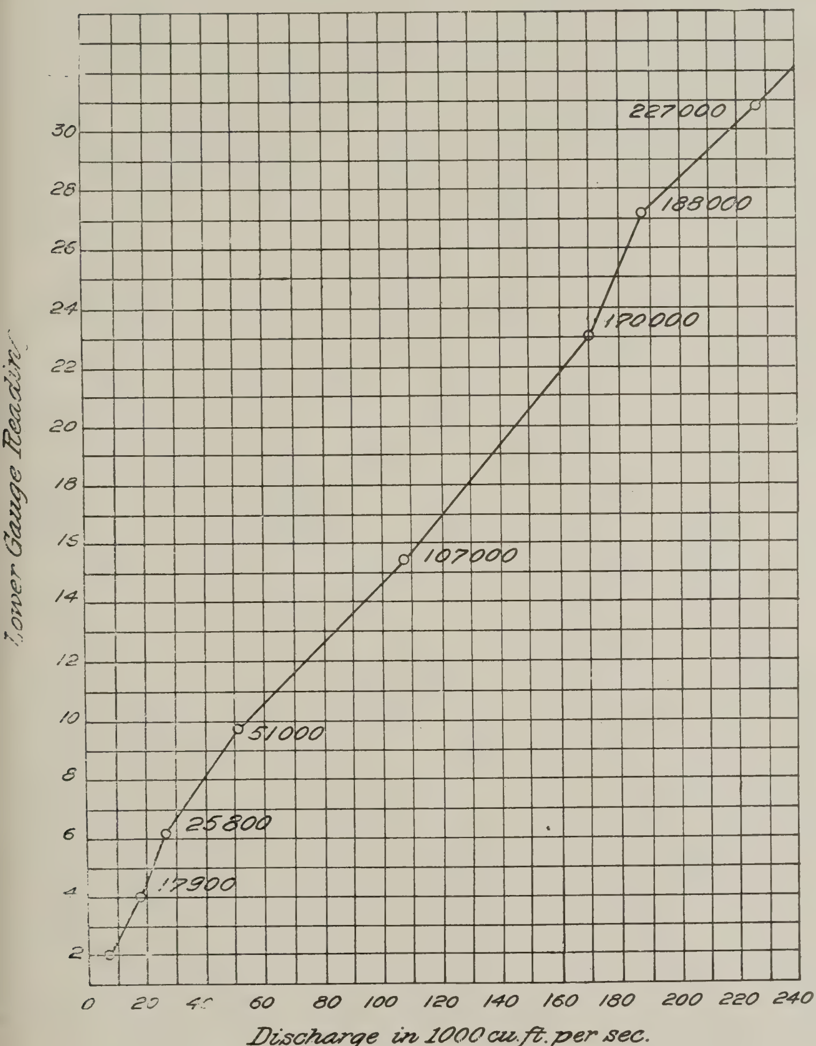
*In charge of Louisville District.

Around the Falls a canal for navigation was built by a private company in 1825-1830. This canal had at its foot a flight of three locks. In 1867-1872 the canal was enlarged and the old locks were replaced by a flight of two locks. Now the General Government is about to enlarge this canal, called the Louisville and Portland Canal, and install a single lock at its foot of the same size, 600 by 110 feet, as are all of the other locks on the Ohio River. A movable dam has been recently built on the crest of the Falls, which furnishes a pool at low water giving a depth of 9 feet over all obstructions to Madison, Ind., 50 miles upstream, the site of the next upstream lock. The normal level of this upper pool is at elevation 412, while the normal level of the lower pool will be at an elevation 383.8. The maximum low-water head will therefore be approximately 28 feet. It will appear later on how this head is diminished and finally almost obliterated during periods of extreme high water.

As this water power possibility lies about in the center of an area composed of the towns of Louisville, New Albany, and Jeffersonville, and supporting a population of 300,000, it has a market immediately at hand for a very large output.

The idea of developing power here is very old. Old mills of small capacity have in the past occupied both sides of the river. In 1873, Mr. Morris S. Belknap read a paper before the American Society of Civil Engineers on this subject which was published in the Transactions of the Society, Vol. II, page 261. As late as 1903 the subject seems to have been thoroughly taken up by Mr. Benzette Williams of Chicago. The outline of the latter's project is in this office. In comment on these two studies it may be sufficient to say that neither of them has been successful in presenting a project in such a light as to attract the necessary capital. It can be seen at a glance in considering Mr. Williams' project that he was so handicapped by conditions imposed by the unsettled nature of the requirements of navigation that it was impossible at the outset.

Now the altered conditions seem to warrant the rediscussion of the subject and these alterations, which appear to tend toward the practicability of the scheme, are as follows: The plans for navigation are definitely settled and well known to the Government officials, and the development of water power has but little bearing on such plans. The United States is about to undertake work at the locality which may be coordinated with the work for water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60
80
100
120
140
160
180
200
220
240
Lower Gauge Reading
Discharge in 1000 cu. ft. per sec.

Fig. 1.

power development to the economical advantage of both, as the cost of each separate piece of work is large and the two will be about equal. Hydroelectric machinery adapted for low heads has been considerably improved in the past few years. The cost of coal seems destined to increase in the future, and likewise the demand for electric power is steadily increasing in the locality. The conditions for navigation have a tendency to force the works for water power to the south side of the river on territory that was formerly occupied by the very valuable cement industry at Louisville. This industry, while of the greatest importance in the recent past, has now been practically abandoned since Portland cement has driven Louisville cement from the field, apparently forever.

It must be borne in mind that this paper is not written by an hydraulic power expert, but the data given are sufficient for any such expert to base sound conclusions upon.

There are not available complete discharge data, but more complete data in this respect are available now than at any time in the past, thanks to the efforts of the United States Geological Survey, and are ample to guide any preliminary investigation. The United States Engineer Department has gauge records covering a continuous period of thirty-five years in the past, and this is ample to cover all requirements in this respect.

Appended hereto will be found a map showing the Falls of the Ohio and the Louisville and Portland Canal, together with the proposed power plant and auxiliaries in outline. Elevations on this map are referred to mean tide at Sandy Hook, N. J. In this discussion the gauge readings pertain to the upper or lower gauge of the present locks of the Louisville and Portland Canal as specified; the elevation of the zero of the upper gauge is 403.004 and of the lower gauge 376.056.

Mention has been made of the movable dam at the head of the Falls. This dam is kept raised at all natural river stages when, without the influence of the dam, the upper gauge would read 9 feet or less, or when the surface of the water on the head of the Falls would naturally be at elevation 412 or less. When the lock next below Louisville shall have been completed its dam will be raised at all natural stages of the river, which would give 7.8 feet or less on the lower gauge of the locks of the Louisville and Portland Canal. The following table shows the conditions as to head by the simultaneous readings of the upper and lower gauges, both with and without the influence of the dams at the head of the Falls and the next one below Louisville.

Simultaneous Gauge Readings Above and Below the Falls.

Dams not in operation.			Dams in operation.		
Upper Gauge.	Lower Gauge.	Head ft.	Upper Gauge.	Lower Gauge.	Head ft.
2	3.0	26.0	9.0	8.0	28.0
3	4.2	25.8	9.0	8.0	28.0
4	5.5	25.5	9.0	8.0	28.0
5	7.0	25.0	9.0	8.0	28.0
6	9.0	24.0	9.0	9.0	27.0
7	11.5	22.5	9.0	11.5	24.5
8	14.7	21.0	9.0	14.7	21.3
9	19.5	16.5	9.0	19.5	16.5
10	24.3	12.7	10.0	24.3	12.7
11	29.0	9.0	11.0	29.0	9.0
12	33.0	6.0	12.0	33.0	6.0
13	35.5	4.5	13.0	35.5	4.5
14	37.4	3.6	14.0	37.4	3.6
15	39.2	2.8	15.0	39.2	2.8
16	40.5	2.5	16.0	40.5	2.5
17	41.7	2.3	17.0	41.7	2.3
18	42.8	2.2	18.0	42.8	2.2
19	43.8	2.2	19.0	43.8	2.2
20	44.8	2.2	20.0	44.8	2.2

The highest recorded flood (1884) reached 46.7 feet on the upper gauge and 72.1 feet on the lower gauge. The lowest known water was 1.7 feet on the upper gauge and 2.0 feet on the lower gauge.

From the gauge record of thirty-five years the following table has been prepared to show the average number of days per year that various stages of the open river exist, readings taken on the lower gauge.

Days of Stages Per Annum (Average Thirty-five Years' Record).

Lower Gauge.	Days per year.	Days below.	Available head ft.	Lower Gauge.	Days per year.	Days below.	Available head ft.
2 to 3	7	---	28	18 to 19	8	251	17
3 to 4	22	7	28	19 to 20	8	259	16
4 to 5	22	29	28	20 to 21	8	267	15
5 to 6	20	51	28	21 to 22	7	275	14
6 to 7	20	71	28	22 to 23	7	282	13
7 to 8	19	91	27	23 to 24	6	289	13
8 to 9	21	110	26	24 to 25	5	295	13
9 to 10	23	131	26	25 to 26	4	300	11
10 to 11	16	154	25	26 to 27	5	304	10
11 to 12	14	170	24	27 to 28	4	309	10
12 to 13	14	184	23	28 to 29	3	312	9
13 to 14	14	198	22	29 to 30	3	315	8
14 to 15	13	212	21	30 to 31	3	318	7
15 to 16	9	225	20	31 to 32	2	321	7
16 to 17	8	233	19	32 to 33	2	323	6
17 to 18	10	241	18				

The following discharge observations are available. No discharge measurements for extreme low water have been made, but 8,000 foot-seconds is assumed.

Discharge Observations (United States Geological Survey).

Lower Gauge	Discharge f. s.	
4.97	17,900	These gaugings were made under the direction of A. H. Horton, District Engineer, United States Geological Survey.
6.22	25,800	
9.66	51,000	
15.45	107,000	
23.18	170,000	
27.12	188,000	
30.68	227,000	

From the above discharge data the following discharge diagram is plotted for readings on the lower gauge. The necessary data for an accurate determination of intermediate discharge is not available nor is it deemed at all necessary in this connection.

From quantities of discharge taken from the foregoing diagram of discharge and the available head at the various gauge readings, the following table of theoretical horsepower available is derived.

Theoretical Horsepower Available.

Lower Gauge.	Discharge f. s.	Head ft.	Horse power.	Horse power minus requirements of navigation
2	8,000	28	25,000	17,000
3	13,000	28	41,000	33,000
4	17,500	28	55,000	47,000
5	22,500	28	71,000	63,000
6	27,000	28	86,000	78,000
7	33,000	27	101,000	93,000
8	40,000	26	118,000	110,000
9	46,000	26	136,000	128,000
10	54,000	25	153,000	146,000
11	64,000	24	175,000	168,000
12	74,000	23	193,000	186,000
13	83,500	22	209,000	203,000
14	93,000	21	222,000	216,000
15	103,000	20	234,000	228,000
16	112,500	19	243,000	238,000
17	120,000	18	246,000	241,000
18	128,000	17	248,000	243,000
19	135,500	16	247,000	242,000
20	140,000	15	239,000	75,240
21	152,000	14	242,000	89,000
22	160,000	13	236,000	94,000
23	169,000	13	250,000	108,000
24	174,000	13	257,000	114,000
25	178,500	11	222,000	102,000
26	183,000	10	208,000	99,000
27	187,000	10	213,000	103,000
28	198,000	9	202,000	104,000
29	210,000	8	191,000	103,000
30	219,000	7	174,000	98,000
31	230,000	7	183,000	106,000
32	240,000	6	163,000	96,000
33	250,000	6	171,000	105,000

It will, of course, be impossible to utilize the full flow of the stream for water power at any stage. For the purposes of navigation through the Louisville and Portland Canal the United States will require at the maximum between stages of 2 feet and 14 feet on the lower gauge 2,500 cubic feet per second. Owing to the fact that when the lower gauge reads about 19 feet coal tows pass down over the falls and no very large volume of water can

then be deflected from the navigable channel, it will not be practicable to develop anything like the power that the theoretical figures indicate for that stage and for stages from that stage up. There is some difficulty in determining the amount of water that must be provided for the navigation of the Indiana Chute at a 9-foot stage. For purposes of rough approximation, it is assumed that when

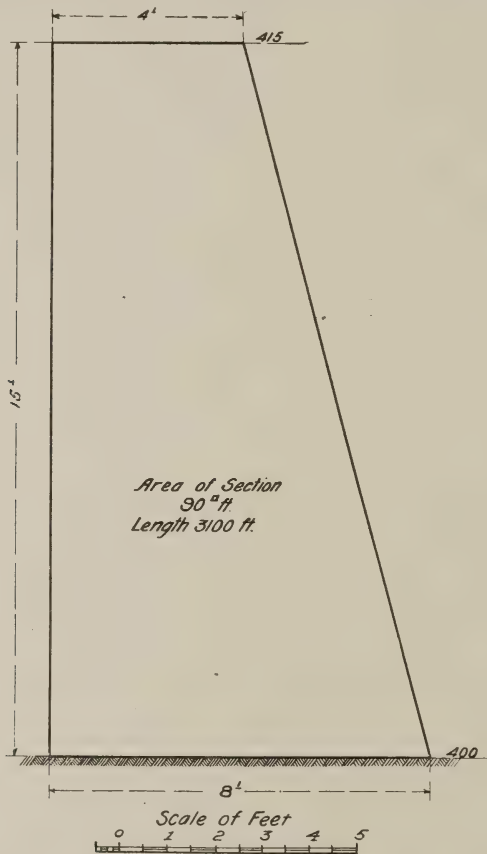


Fig. 2. Average section of dike between United States Dam and Pennsylvania Bridge. This section is much heavier than the full 15-foot head requires and is made so in view of necessary resistance to heavy ice that tends toward Indiana Chute and which will take wall at crest.

the movable dam is lowered, 70 per cent of the discharge at that stage must pass down the chute in order that the water surface shall not be lowered below what is the case now. This assumes that the fore bay for water power is located as is shown on the

OHIO RIVER
WILLOW POINT
STONE DIKE ELEV 395
STONY POINT
STONE DIKE ELEV 400
WAVE ROCK
GOOSE ISLAND
MIDDLE CHUTE
ROCK ISLAND
PROPOSED CONCRETE DIKE ELEV 415
PROPOSED MOVABLE BOULE DAM – SILL 405 CREST 415
KENTUCKY CHUTE
INDIANA CHUTE
OLD MILL RACE
LOUISVILLE BRIDGE CO.
PROPOSED CONCRETE DIKE ELEV 415
FIXED CONCRETE DIKE ELEV 412
MOVABLE DAM
CHANOINE DAM
MOVABLE BOULE DAM
Proposed Removal
FIXED CONCRETE DIKE EL 411
POWER HOUSE DEVELOPMENT
AUXILIARY STEAM PLANT
SAND ISLAND DAM
SHIPPINGPORT
PROPOSED COAL RAIL LINE
U.S. PROPERTY LINE
LOUISVILLE & PORTLAND CANAL
STONE WALL ELEV 418
OLD LOCKS
NEW LOCKS
SAND ISLAND
HIGH AVE
PORTLAND AVE.
WATER STREET
MAIN STREET
CITY OF LOUISVILLE
POSSIBILITIES OF POWER DEVELOPMENT.
AT
The Falls of the Ohio, Louisville, Kentucky
PREPARED BY
Major Lytle Brown, C.E.U.S. Army
Scale Of Feet
0 500 1000 1500

accompanying map. This will require a discharge of 96,000 cubic feet in the Indiana Chute at this stage, and this quantity will diminish as the water rises on the lower gauge. This quantity, however, will be used in the preparation of the final table of actual horsepower available. In connection with this feature of tow-boat navigation over the Falls it may be taken that, since the United States will shortly increase greatly the capacity of the Louisville and Portland Canal, the importance of the navigation over the

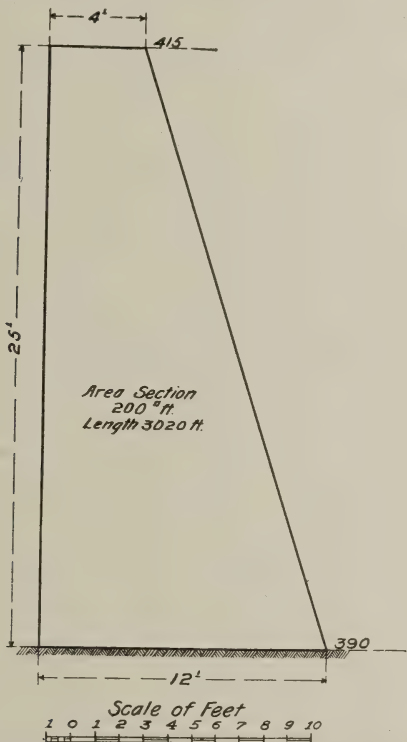


Fig. 3. Average section of dike between Pennsylvania Bridge and Ware Rock. This section is much heavier than the full 25-foot head of water requires and is made so to resist the shock of heavy ice striking near the crest.

Falls at all stages less than 12 feet on the upper gauge will be greatly reduced. It is also to be considered probable that the handling of very large tows on the Ohio will diminish when once the stream is completely canalized.

In considering the actual horsepower that may be expected at the dynamo terminals the efficiency of the dynamo is taken at 90

per cent and that of the turbines at 70 per cent, these figures being intended to roughly cover the average conditions. The question of water leakage will cut a small figure at periods of the lowest discharge, but, as the dams can be made practically water-tight, this feature is entirely ignored.

The following table is drawn to show the horsepower that may be delivered at the terminals in electrical energy for the various gauge readings and the corresponding days per year when such power may be expected.

Actual Horsepower at Dynamo Terminals.

Lower Gauge.	Horsepower.	Head.	Days per year.	Lower Gauge.	Horsepower.	Head.	Days per year.
2	10,700	28	7*	18	153,000	17	8
3	20,700	28	22	19	152,000	16	8
4	29,600	28	22	20	47,400	15	8
5	39,600	28	20	21	56,000	14	7
6	49,100	28	20	22	59,200	13	7
7	58,500	27	19	23	68,000	13	6
8	69,300	26	21	24	71,800	13	5
9	80,600	26	23	25	64,200	11	4
10	92,000	25	16	26	62,300	10	5
11	105,800	24	14	27	64,800	10	4
12	117,000	23	14	28	65,500	9	3
13	127,800	22	14	29	64,800	8	3
14	136,000	21	13	30	61,700	7	3
15	143,600	20	9	31	66,700	7	2
16	149,900	19	8	32	60,400	6	2
17	151,800	18	10	33	66,000	6	----

The amount of power to be developed will depend somewhat on the availability of space for, and the cost of, the turbine units. As it is desirable to develop as much power as is practicable at one turbine unit, both for saving in power-house space and in electrical equipment, either a duplex or triplex turbine should be installed in each case as a unit. From the number of days that each particular available head prevails it would appear that these units should develop their full power at a head of about 20 feet, or between 14 and 20. It is practicable to obtain such a set of vertical duplex turbines to develop 1,000 horsepower for each set, the distance between centers of shafts being properly about 32 feet.

*Gauge read from 2.0 to 2.99 for seven days; from 3.0 to 3.99 for twenty-two days, etc.

Fifty such sets would develop a front of power-house of 1,600 feet, which is perfectly practicable at this site, as will be apparent on inspecting the general lay-out. The combined total of water consumption would be about 120,000 foot-seconds at 20-foot head. This would, with the fore bay as indicated, cause no appreciable drawing down of the water in the fore bay. As all tail races will discharge directly into the open river there need be no fear of loss of energy due to restricted area at these points. The amount of power that should be provided for in the water-power installation

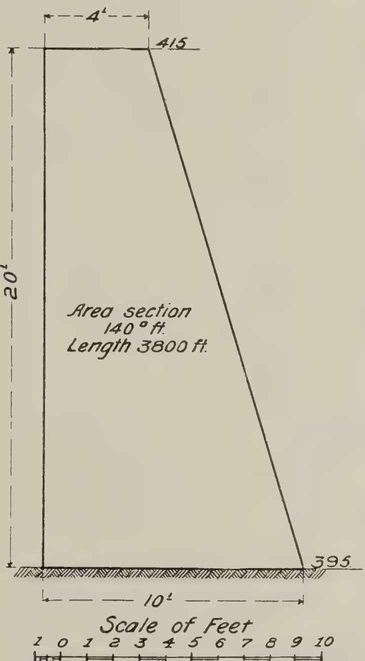


Fig. 4. Average section of dike between Ware Rock and Rock Island. Same remarks as for previous sections as to cross section.

should at the outset equal the demand for power in the city of Louisville and adjacent territory, and provision should be made so that future expansion may take place. It appears at a general glance at the foregoing figures that 75,000 horsepower would be the limiting maximum. Any development of water power should look toward a complete supply of the market. Eliminating electric railway lines, this would demand fully 50,000 horsepower and that figure is taken as the amount to be considered.

Turning to the general lay-out of works, the following features are noted: For the creation of a fore bay a fixed concrete dike is proposed. This dike starts from the north pier of the south section of the Boulé dam of the United States and runs on a rock bottom in a straight line to the fourth pier south of the main channel span of the Pennsylvania Railroad bridge, 3,100 feet; thence in a straight line on rock bottom to the south limit of the United States channel excavation on Wave Rock, a distance of 3,020 feet. At this point, and running nearly west and at right angles to the

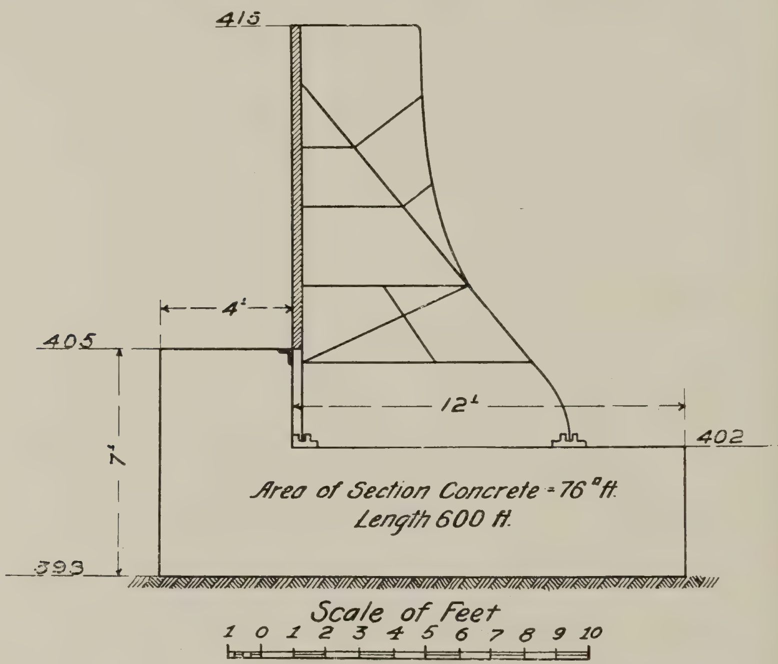
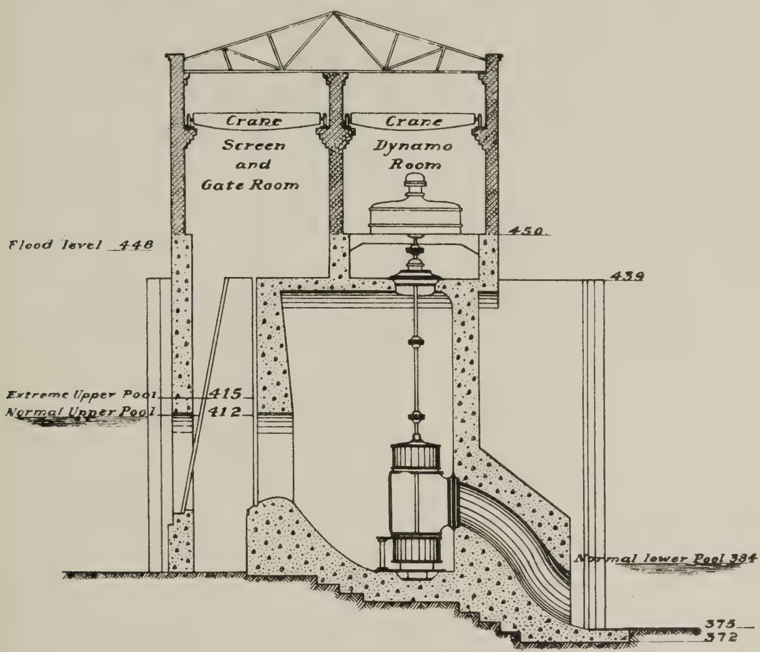


Fig. 5. Average section of Boulé Weir. Steel furnished by the United States.

fixed concrete dike, it is proposed to install a section of Boulé dam from the portion belonging to the United States at the head of the fore bay, which it is proposed to remove. This weir is 600 feet long with elevation of crest at 415 and sill at 405. The object of this weir is to afford upstream navigation when the lower gauge reads over 33 feet, at which time the available head for water power is less than 6 feet and the canal will be closed to navigation. This dam will, of course, be operated by the United States. From the west end of this Boulé dam the fixed concrete dike runs to Rock

Island on a rock bottom, and there terminates at the power house, this last section being 3,800 feet long. The top of the fixed concrete dike and the movable Boulé dam are both at the constant elevation 415.

The power house development is laid out in a broken line fol-



Typical Cross Section of Power House.

Scale of Feet

5 0 25 50 75

Thickness of Cross Division Piers = 6 ft.

Thickness of Lower Longitudinal Walls = 6 ft.

Fig. 6.

lowing such direction as will fit the shore line and minimize excavation without throwing construction into deep water. Provision is made in the outlay for fifty 1,000-horsepower units, allowing 40 feet as distance between centers. They can, doubtless, be placed a little closer. There remains along the shore line, before

reaching the exit of the old locks of the Louisville and Portland Canal, space for the installation of twenty-five additional units.

In order that the same force may operate both the water power plant and the steam auxiliary plant, the latter should be close at hand. A location is shown across the fore bay for the auxiliary steam outlay at a point where coal may be conveniently delivered by barge or rail.

The highest known flood (1884) reached an elevation of 448 at the site of the power house, and it is thought that both water power and steam power houses should have their machinery floors at elevation 450 to insure the protection of all machinery and the continuity of service.

The general character of the fore bay and power house construction are shown in the illustrations.

The following estimate, which is considered as liberal, gives the cost of the water power installation.

Concrete Dike of Fore Bay.

United States dam to Pennsylvania Railroad bridge, concrete, cu. yds.	10,333
Pennsylvania Railroad bridge to Wave Rock, concrete, cu. yds.	22,370
Movable weir, concrete, cu. yds.	1,700
Total, concrete, cu. yds.	34,403
Concrete, 34,403 cubic yards at $6.00	$206,418
Dismantling and reconstruction, 600 feet Boulé weir at $30.00	18,000
Total	$224,418
Say	$225,000

Power House Foundation and Wheel Pits.

Concrete, 195,000 cubic yards at $8.00	$1,560,000
Rock excavation, 50,000 cubic yards at $2.00	100,000
Earth excavation (foundation and fore bay) 150,000 cu. yds. at 25c.	37,500
Power house, 5,000,000 cubic feet at 15c.	750,000
Gates, screens, etc.	50,000
Turbo-electrical equipment, 50,000 horsepower at $23.00	1,150,000
Land	50,000
Total	$3,922,500
Ten per cent contingencies and engineering	392,250
Total	$4,314.750

The above estimate is crude, and would be considerably altered by detailed information of foundation conditions and carefully made plans, but an effort has been made to keep it above rather than below what may be expected. It is thought that close study

by an expert and more detailed information would result in lower figures.

Assuming the above figures for the cost of the water power installation, the following outline of the economical phase of the question may be drawn. It is assumed that the full capacity of the water power end of the plant will, on the average, be realized for two hundred and twenty-five days per year, as is indicated in previous tables. It is further assumed that the same personnel will be used in the water power plant as is used in the steam plant. It is assumed that $25,000 per year is charged against the water power plant for dredging in the fore bay. So that the only offset

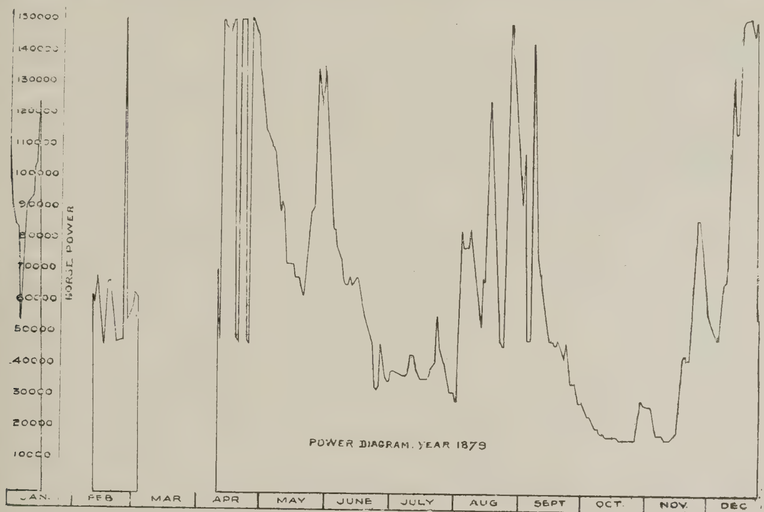


Fig. 7. Possible horsepower at terminals shown graphically for 1879.

against the steam plant for all-the-year operation is the cost of fuel for 225 days.

As a result of the above assumptions and assuming that the steam plant can be erected complete at a cost of $75 per kilowatt, or a total of $2,850,000; and further assuming that the plants are operated twenty-four hours per day, the following comparison results:

Combined Plant.

Interest on installation at 6 per cent	$429,885.00
Depreciation and repairs, water power, at 3 per cent	129,500.00
Depreciation and repairs, steam, at 3 per cent	84,000.00
Dredging in fore bay	25,000.00
Fuel, 140 days at $13.00 per horsepower	249,315.00
Total	$917,700.00
Cost per horsepower, exclusive of labor	18.354

Steam Plant.

Interest on installation at 6 per cent	$171,000.00
Depreciation and repairs at 5 per cent	142,500.00
Fuel at $13.00 per horsepower (365 days)	650,000.00
Total	$963,500.00
Cost per horsepower, exclusive of labor	19.27

The figures would not show in favor of the combined plant if the items on depreciation and repairs were not assumed as being favorably affected toward the combined plant by virtue of non-use of plant for periods of time represented by eight months in case of the steam end and four months in the case of the water power end.

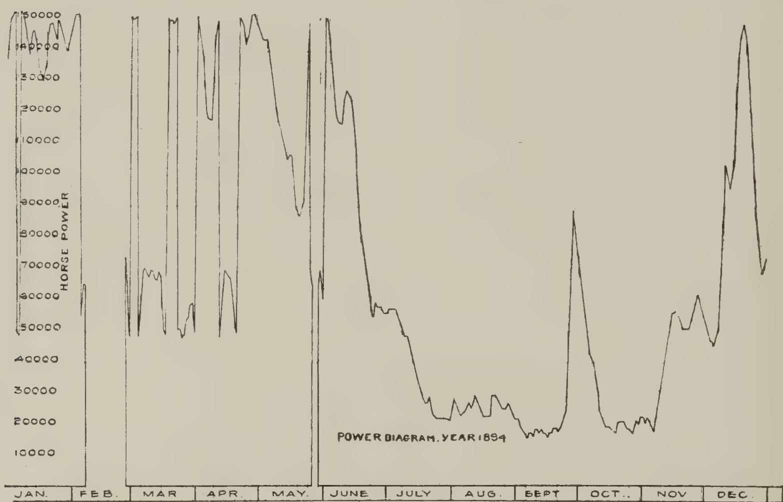


Fig. 8. Possible horsepower at terminals shown graphically for 1894.

Nor is it probable that at the present time the full 50,000 horsepower could be utilized throughout the twenty-four hours of the day.

The following advantageous features may be noted in connection with the location of this steam plant and the water power scheme. The bringing of fuel to the plant by water or rail; river water for condensing purposes. The placing of a steam plant on property that is at present of very small value. The removal from the city of a large part of the smoke nuisance. The possibility of extending for 2 miles the city's navigable water front, and by the necessary dredging of the fore bay, the building up above flood level of a large tract of land that is now useless for industrial purposes on account of floods. A hydraulic dredge maintained for the neces-

sary dredging in the fore bay will build up this land in course of time, and so afford to the city of Louisville an ideal location for manufacturing sites, available to both rail and the improved Ohio River.

One feature of the combined plant will require consideration: the transfer from water power to steam power and vice versa, following fluctuations in the water level. The height and duration of flood stages in the river can not be predicted with certainty, especially the duration. But that heights will prevail that will affect the operation of the water power plant can be certainly predicted for several days in advance. Stages can now be predicted

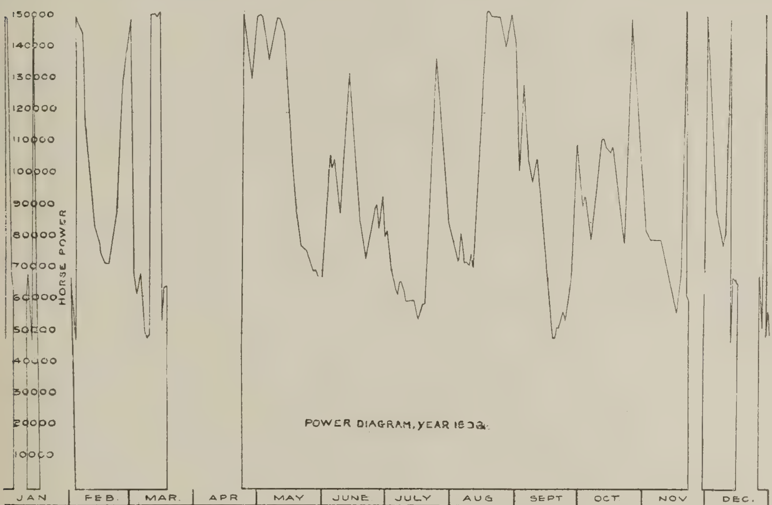


Fig. 9. Possible horsepower at terminals shown graphically for 1906.

with great accuracy and will be predicted with greater accuracy in the future. With past records of gauge heights and rainfall the probable duration of stages can be stated with some confidence for any stated conditions. It would seem that no embarrassment should ever occur in swinging the steam plant into and out of operation.

The power possible of attainment at the dynamo terminals is shown graphically for each day of the three years, 1906, 1894, and 1879, expressed in horsepower. In plotting these three power curves heads less than 12 feet were eliminated, the average efficiency of the turbines over a range of head from 12 feet to 28 feet was assumed to be 70 per cent and that of the dynamos 90 per

cent, or the power as plotted is taken as 63 per cent of the theoretical power available. These power curves were chosen from similar curves for each year, 1876 to 1910, inclusive, and of all these years 1906 is considered a favorable one, 1894 and 1879 as unfavorable or, at most, average ones. The diagrams show the following approximate results.

	Days of head above 12 feet.	Days of 50,000 hp. and over.
1906	289	282
1894	340	208
1879	298	174

It is to be noted that the scheme for the dikes of the fore bay is such as will be of decided benefit to navigation descending the Indiana Chute, and some such location would be insisted on by the Government's authorities.

Rapid Cost Estimation for Piers and Breakwaters

BY

Mr. G. A. M. Liljencrantz
Assistant Engineer

Orders from the Department in Washington are liable to be received at any time for estimates of the probable cost of some piers or breakwaters, and these estimates are generally to be submitted "as soon as possible."

To prepare such estimates—based on structures suitable for various depths of water, for different exposures to strong winds and for local conditions—from available plans and tables of materials, involves a considerable amount of work, with many chances for making errors, especially when this work has to be done in haste; and the writer conceived many years ago a method by which the desired results may be obtained in a very short space of time and with a degree of accuracy which will fully answer the requirements in such cases, as far as the amounts of materials are concerned. Provisions were made for the adaptation of governing prices for materials in the localities, and for the period, where and when a proposed work is to be done.

This method, with tables, diagrams, etc., was described and embodied in the Report of the Chief of Engineers for the year ending June 30, 1896; but, in making use of these data lately, the shape in which they were there presented was found to be unsatisfactory in several respects, as follows:

1. The principle on which the method is based is not clearly shown.

2. The constants which are to be used in the formula are scattered too much for convenient use.

3. Four different variations of the method were suggested with a view to securing that many different degrees of accuracy. This is confusing and unnecessary, as the method giving the most accurate results is simple and should be used exclusively.

4. It is deemed desirable to maintain uniformity as to the di-

mensions of at least the principal parts of the structure, which was not done in all cases.

5. Corrections of some of the data were also found necessary. I have therefore revised the whole subject and arranged it in such a shape that it can be used easily, quickly and safely, without anything more being required than a perusal of the first division of this paper.

The whole subject has been divided into two principal sections or divisions, viz:

a. Containing a description of the principles on which the method of procedure is based; the formula to be used in making the computations and a table giving the values of the constants used in the formula.

b. Containing various explanations, verifications, modifications, and additions, all of which may prove useful to anyone who may wish to satisfy himself as to the accuracy of the method, or who desires to make use of more detail information relative to modified forms in the structures, and to auxiliary work work which may be used in connection with the structures described.

a. BASIS FOR, AND USE OF, THE FORMULA.

1. *Types of Structures*. There are six different types of structures considered in this paper, all of which have been used in the Chicago District, and each of which consists of a crib construction, 100 feet long.

They are as follows:

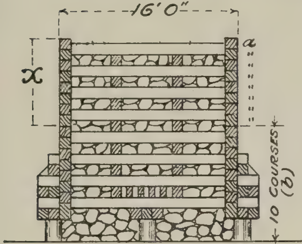
Type No. 1. Cribs 16 feet wide on pile foundation.
Type No. 2. Cribs 20 feet wide on pile foundation.
Type No. 3. Cribs 24 feet wide on pile foundation.
Type No. 4. Cribs 30 feet wide on pile foundation.
Type No. 5. Cribs 30 feet wide on stone foundation, 4 feet high.
Type No. 6. Cribs 30 feet wide on stone foundation, 6 feet high.
(See Plate I.)

2. An examination of the various plans will show that in *each* of the above types the amount of materials contained in the ten lower courses (counting from the lake bottom) is a constant quantity for that type; and that in every two successive courses above the tenth course the quantities are the same for each type, respectively. (See Plate I.)

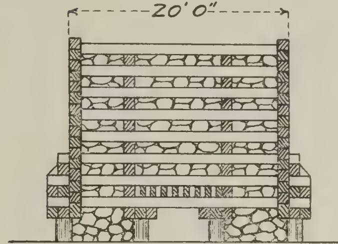
3. This fact suggested the practicability of using the formula for a straight line (in a plane) for the computation of the total cost of

PLATE I

CROSS SECTIONS OF REPRESENTATIVE CRIB STRUCTURES.



TYPE 1



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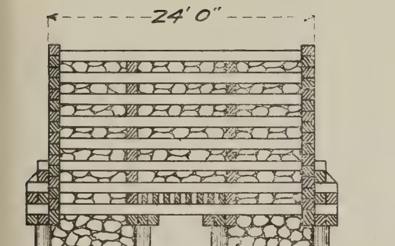

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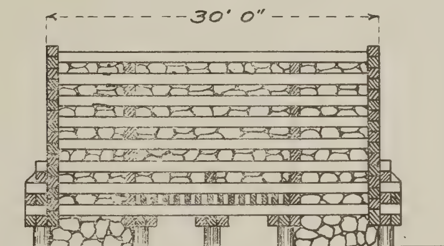

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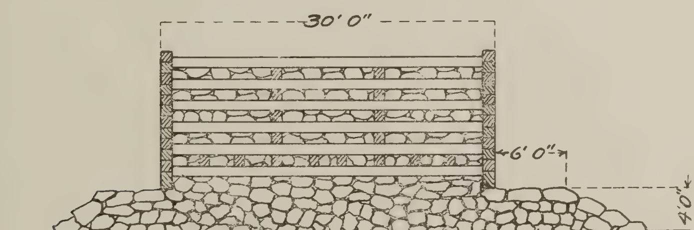
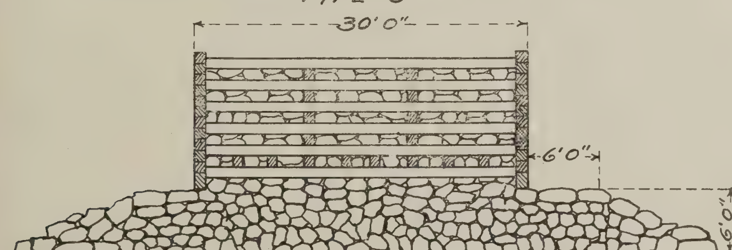


TYPE 5



TYPE 6.

a crib of any desired height, after the materials in the ten lower courses, and in two of the upper courses, respectively, have been ascertained. Thus we have,

$$Y = aX + b. \qquad (1)$$

in which Y represents the cost of a crib structure 100 feet long, according to either of the types; a represents the cost of all materials in one of the upper courses (the constants given in the table below being the average of two of the upper courses). X equals the number of the courses desired *above* the tenth course; and b equals the cost of all materials used in the lower ten courses. The height of the timber work above the lake bottom (2 feet in types 1 to 4, inclusive) is counted as two courses. The stone foundations, in types 5 and 6, are counted as four and six courses, respectively. The bottom course of the cribwork is 1.5 feet high; all other courses 1 foot each.

4. There is one discrepancy in the formula, that would affect the accuracy of results if not remedied. The error and its remedy will be accounted for later on. (See paragraph 7.)

5. *Use of the Formula.* As stated above, the constants a and b represent the cost of *all* materials in the different parts of the crib, as noted. These materials consist of timber, drift bolts and stone and (in types 1 to 4, inclusive) of piles and screw bolts in constant b. The unit prices of each of the materials will also enter as factors. Thus the formula, containing all these items, will be as follows:

$$Y = (a^t T + a^d D + a^s S) X + b^t T + b^d D + b^s S + pP + cC \qquad (2)$$

in which a^t and b^t, a^d and b^d, a^s and b^s, p and c represent, respectively, the quantities of timber, drift bolts, stone, piles and screw bolts* (as given in Table I below) for each type, and T, D, S, P and C the unit prices for materials, viz, per thousand feet board measure for timber; per hundredweight for bolts; per cord of 128 cubic feet for stone, and for each of piles. All prices for materials are for these "secured in the work." It must be particularly remembered that X represents the number of courses *above* the tenth course.

It may be found desirable, in preparing an aggregate bill of ma-

*The screw bolts are used for fastening each set, respectively, of the bottom bearing timbers together.

terials for a proposed work, to ascertain quickly the amounts of these materials. While a study of the formula (2) would suggest how this information may be easily obtained, it may not be amiss to point out the fact here.

As has already been stated, the constant a^t represents the amount of timber required in *one* of the courses above the tenth course, and the constant b^t the total amount in the ten lower courses. From this it follows clearly that the total amount of timber required in 100 linear feet of a crib of any of the types, respectively, will be found by means of the simple formula:

$$Y^t = a^t\ X + b^t,$$

using the respective values for the constants in Table I.

In the same manner the amount of drift bolts, stone, etc., may be obtained.

The constants to be used in the formula are given in the Table I.

Table I. Constants for types with ties and longitudinals of 10×12-inch timbers.

No.	Types.		a^t	a^d	a^s	b^t	b^d	b^s	p	c
	Width of Crib.	Foundation.	Thou. ft. Board Measure.	Cwts.	Cords.	Thou. ft. Board Measure.	Cwts.	Cords.	No.	Cwt
	Feet.									
1	16	Pile.	4.680	4.3	9.605	56.648	37.0	92.947	27	5.1
2	20	Pile.	4.996	4.4	12.525	64.894	37.5	100.450	40	6.7
3	24	Pile.	5.312	4.4	15.444	67.734	37.8	129.336	40	6.7
4	30	Pile.	5.786	4.6	19.823	79.518	38.3	150.641	53	8.4
5	30	Stone	5.786	4.6	19.823	40.492	25.9	259.943	0	0
6	30	Stone	5.786	4.6	19.823	28.920	17.3	301.547	0	0

The discrepancies referred to above (in paragraph 4) have been remedied (as will be explained later on in paragraph 7) and the constants have been corrected accordingly and are to be used *as given* in the table.

b. EXPLANATIONS, MODIFICATIONS, AND ADDITIONS.

6. *Chief Elements in the Structures.* It may be found desirable to verify the various amounts entering into the calculation, and for that purpose the general dimensions of timbers, etc., are here given; it being believed that, for the sake of comparison between the different types, it is desirable to maintain uniformity with regard to the dimensions of all the principal parts of the structures.

Thus the following dimensions have been used for each of the six types:

Bottom side timbers, 12 by 18 inches.

All other side timbers, as well as end timbers and bearing timbers, 12 by 12 inches.

Ties and longitudinals, 10 by 12 inches (2-foot long scarves having been provided for the latter).

Stone bottom (in types 1 to 4) in middle pockets, 6 by 12 inches. The grillage bottom in types 5 and 6 are made of 12 by 12 inch timbers.

The lowest set of longitudinals (in types 1 to 4) are extended to the full length of the crib, 100 feet, and blocks are placed at each end of the crib, between these longitudinals and the bearing timbers, which are also 100 feet in length. An extra bearing timber, 12 by 12 inches, is provided—in type 4—above the stone bottom. (See Plate I.)

All cross ties are dove-tailed into the side timbers and the longitudinals into the end walls.

Stone has been provided for in the wells between the cribs, the calculation thus covering the filling of the crib for its entire length.

In estimating the amount of stone required, calculation has been made on the assumption that the whole volume of the crib, less the space occupied by timber, is filled with stone. While this is not strictly correct, it has been done to compensate for such stone that will usually settle down into the sand bottom and work out on the sides; also, to some extent, for irregularity in the lake bottom.

Drift bolts, 32 inches and 20 inches in length, are provided for in regular columns in the side and end walls; also through bearing timbers and protruding ties, and in the crossings of ties and longitudinals, alternately in each crossing.

7. *Discrepancy in the Formula and its Remedy.* It was stated above that the quantities in each set of two courses above the tenth course are constant amounts for each type. There is an exception to this rule. The top course of every structure has less timber and stone than each of the other courses, because there are no longitudinals in this course and no stone in the upper half of it. There are, however, more bolts in the top course, as a 20-inch bolt is placed in each of the top, side and end walls, in each column of 32-inch bolts, where these reach only to the upper face of the third course from top.

To remedy this, the respective amounts are deducted—for timber and stone—and added for drift bolts, to the corresponding materials in the constants b, for each respective type. These amounts are given in Table III, for a special purpose, as will be accounted for later on. (See paragraph 9.) The figures given in Table I are the *corrected* amounts, as already mentioned.

8. *Suggested Simplification.* If a large number of cribs of different types and heights are to be estimated for, and prices suitable to the locality and period have been determined, the formula may be materially simplified by inserting these prices in the formula and developing the calculations for the values of a and b, respectively, thus giving the formula the initial form:

$$Y = aX + b$$

after which the computation will be reduced to the simplest kind, according to the variations in the values of X.

Thus, for an example, in making some estimates lately for the cost of cribwork in this district, the following prices were made use of:

For timber, $50.00 per thousand feet board measure.
For drift bolts, $3.00 per hundred-weight.
For stone, $7.00 per cord of 128 cubic feet.
For pine piles, $12.00 each; and
For screw bolts, $4.00 per hundred-weight.

Assuming a case in which the above-quoted prices are acceptable, both as to period and locality, then the formula developed in accordance therewith will give, for each of the types herein considered, the simplified formulas, as follows:

For type 1: $Y = 314.54 \times X + 3{,}938.43$.
For type 2: $Y = 350.68 \times X + 4{,}567.15$.
For type 3: $Y = 386.91 \times X + 4{,}894.25$.
For type 4: $Y = 441.86 \times X + 5{,}814.89$.
For type 5: $Y = 441.86 \times X + 3{,}921.90$.
For type 6: $Y = 441.86 \times X + 3{,}608.73$.
For type 1A: $Y = 331.78 \times X + 4{,}076.12$.
For type 2A: $Y = 370.31 \times X + 4{,}720.84$.
For type 3A: $Y = 408.55 \times X + 5{,}069.40$.
For type 4A: $Y = 466.10 \times X + 6{,}007.67$.
For type 5A: $Y = 466.10 \times X + 4{,}051.37$.
For type 6A: $Y = 466.10 \times X + 3{,}688.95$.

To facilitate a convenient comparison between the cost of the different types, the above formulas have been developed for a uniform height of 22 courses (making X=22—10=12).

No. of type.	Cost of 100 linear feet.	Cost per linear foot.
1	$7,712.91	$77.13
2	8,775.31	87.75
3	9,537.17	95.37
4	11,117.21	111.17
5	9,224.22	92.24
6	8,911.05	89.11
1A	8,057.48	80.57
2A	9,164.56	91.65
3A	9,972.00	99.72
4A	11,600.87	116.01
5A	9,644.57	96.45
6A	9,282.15	92.82

The diagram (Plate II) will furnish means of readily comparing the relative cost of the different types of cribs, all of these being calculated for the same height (22 courses) above the lake bottom.

9. *Different Quality of Timber in Sub- and Super-structure.* The above formulas contemplate only one kind of timber for the whole structure. Should it be required to provide for different quality of timber for sub-and super-structure (where the prices differ materially) then proceed as follows:

a. For the Sub-structure. Use the general formula (1) with constants as given in Table I; adapting the prices for timber to be used, and with X fixed according to the number of courses in *this part* of the work.

b. For the Super-structure. Use the same formula with the constant a as above; but for the constant b substitute the values for this constant (with + or — signs as indicated) as given in Table III, and make X=the number of courses desired in the super-structure; and use the price for the proposed kind of timber, accordingly.

PLATE Z.

DIAGRAM SHOWING RELATIVE COST OF REPRESENTATIVE CRIB STRUCTURES.

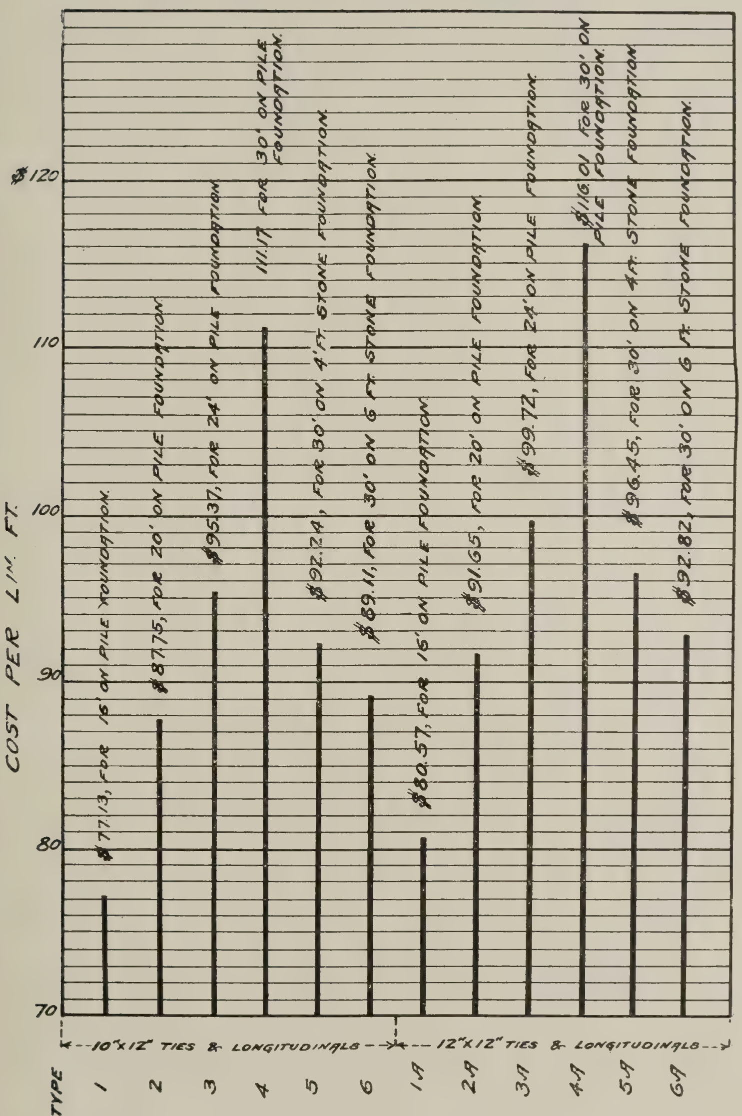


Table III. Constants used to compensate for discrepancy in the top courses.

Types.			With 10 by 12 inch timbers.			With 12 by 12 inch timbers.		
No.	Width of crib.	Foundation.	Timber, 1,000 feet B. M.	Bolts. Cwts.	Stone. Cords.	Timber, 1,000 feet B. M.	Bolts. Cwt.	Stone. Cords.
	Feet.							
1	16	Pile.	—0.960	+2.3	—4.747	—1.152	+2.3	—4.625
2	20	Pile.	—0.960	+2.4	—6.279	—1.152	+2.4	—6.157
3	24	Pile.	—0.960	+2.4	—8.591	—1.152	+2.4	—7.687
4	30	Pile.	—0.960	+2.5	—10.107	—1.152	+2.5	—9.984
5	30	Stone	—0.960	+2.5	—10.107	—1.152	+2.5	—9.984
6	30	Stone	—0.960	+2.5	—10.107	—1.152	+2.5	—9.984

10. *Rebuilding Super-structure.* Should it be required to prepare an estimate of cost of rebuilding the super-structure over old crib work, proceed in the same manner as described in case (*b*) paragraph 9, adding only the estimated cost of removing the old work.

11. *Modification in Types.* In some districts, it may be preferred to use 12 by 12 inch timbers for ties and longitudinals. This would increase the constants a^t and b^t for timber, and decrease the constants a^s and b^s for stone. The constants for bolts would not be affected. Table II below gives the constants calculated to fit such cases; all other elements remaining unaltered.

Table II. Constants for types with ties and longitudinals of 12×12 inch timbers

Types.			a^t 1,000 ft. B. M.	a^d Cwt.	a^s Cords.	b^t 1,000 feet B. M.	b^d Cwt.	b^s Cords.	p Piles.	c Sc'w bolts
No.	Width of crib.	Foundation.								
	Feet.									
1A	16	Pile.	5.064	4.3	9.383	59.622	37.0	91.374	27	5.1
2A	20	Pile.	5.424	4.4	12.273	68.220	37.5	98.648	40	6.7
3A	24	Pile.	5.784	4.4	15.164	71.052	37.8	128.086	40	6.7
4A	30	Pile.	6.316	4.6	19.500	83.706	38.3	148.267	53	8.4
5A	30	Stone	6.316	4.6	19.500	43.320	25.9	258.238	00	0.0
6A	30	Stone	6.316	4.6	19.500	30.672	17.3	300.493	0	0.0

12. *Tables for Other Types of Cribs.* Some districts have types of crib work differing more or less from those herein described, and which are there preferred. Should it be so desired, tables of constants may easily be prepared for such types on the same prin-

ciple and the use of the same formula, with the constants modified to fit those structures.

It may be said that the constants will not apply to other types of crib work, which is true. Still, I believe that in most cases the total amounts will be so nearly similar that an addition or deduction, as the case may be, of a comparatively small percentage will give a sufficiently accurate result for any preliminary estimate.

13. *Decking.* The crib work is generally covered by some kind of decking to protect the stone filling, especially if the work is exposed to severe storms. The form of decking generally used in this district consists of 6 by 10 inch planking laid flat, 2 inches apart, and spiked to the cross ties with ½ by 14 inch spikes, washers being used under the heads of the spikes, which has proved very advantageous in giving much better hold. One hundred linear feet of a deck plank contains 500 feet board measure (=0.5 thousand feet board measure) and the spikes required for that length of deck plank, including washers, will weigh approximately 50 pounds (or =0.5 hundred-weight).

The total cost of the decking (of the kind described) for any of the types of cribs may be obtained by the simple formula:

$$Z=\frac{(E+K)\,X}{2} \qquad (3)$$

in which Z represents the cost of the decking over 100 linear feet of the crib; E=the cost per thousand feet board measure of the deck timbers; K=the cost per hundred-weight of the spikes and washers, and X=the width (*inside* of the side timbers) of the crib to be covered.

14. *Intermediate Decking Supports.* When a pier or breakwater is greatly exposed to severe gales, it is very desirable to place intermediate supports under the decking, half-way between the cross ties. These supports may be made of 3 by 12 inch planks placed on edge and resting on the two top longitudinals and, at each end, on a 3 by 12 inch by 2-foot piece of plank spiked to the side walls, with their tops level with the tops of the longitudinals. The length of each plank will be = the inside width of the crib. If, however, they are made (in the estimate) = the *outer* width of the crib + 2 feet, in length, the two pieces to be spiked to the side walls will be provided for. The decking should, as a matter of course, be spiked to this planking. As there are twelve spaces between the ties in

each crib, the cost of the decking supports will be obtained by the formula:

$$Y = 0.036\ F\ (w+2) + 0.3\ K\ (w-2) \qquad (4)$$

in which Y represents the total cost of the decking supports for a crib 100 feet long; F=the cost per thousand feet board measure of the planks; K=the cost per hundred-weight of spikes and washers and w=the *outside* width of the crib.

In conclusion, I beg leave to add that it will be very much appreciated if anyone finding errors, or the appearance of errors, in any part of the foregoing, will kindly so inform me, so that I may have a chance to either correct such errors, if existing, or make an explanation of any apparent errors where none really exist.

Cantilever System of Concrete Forms, Mayos Bar Lock, Coosa River, Georgia

BY

Mr. Thomas J. Kelly
Overseer

The accompanying sketch and illustration show the cantilever system of concrete forming that is being used on lock walls at Mayos Bar, Coosa River, Georgia. The first section is a stationary form, which is braced from the ground, the concrete being put in in lifts of 6 feet each. In the second and succeeding lifts, bolts ¾ inch by 30 inches with nuts on each end are put in the concrete 10 inches above the bottom and 10 inches below the top of the lift, and 4 feet apart, leaving enough of the bolts outside of the concrete to go through the lagging, uprights and wales.

When the concrete has sufficiently set, the lower form is removed and holes bored in the wales to correspond with each bolt. The wales are then put on the bolts and uprights are put in between the wales and concrete, first putting one piece of lagging on top of the upper bolts, next to the face of the concrete; wedges are then driven near bottom end of the uprights, between the face of the concrete and the uprights, these wedges being used to line the forms and keep them rigidly in place.

The bolts should be left outside the forming, as shown in the upper left hand of Fig. 1, until just before the concrete reaches this point; they should then be greased with "cable dope" and shoved in to the proper position.

Sometime during the next day, the bolts should be given a few turns with a wrench, so they will be loose in the concrete and can be removed at any time, and the holes plugged with neat cement. Nailing strips, 2 by 4 inches, are used to tack the lagging together, and a strip of the same size is also used to secure the bulkhead, or end forming, in place.

After the form has been used the first time, the lagging can be

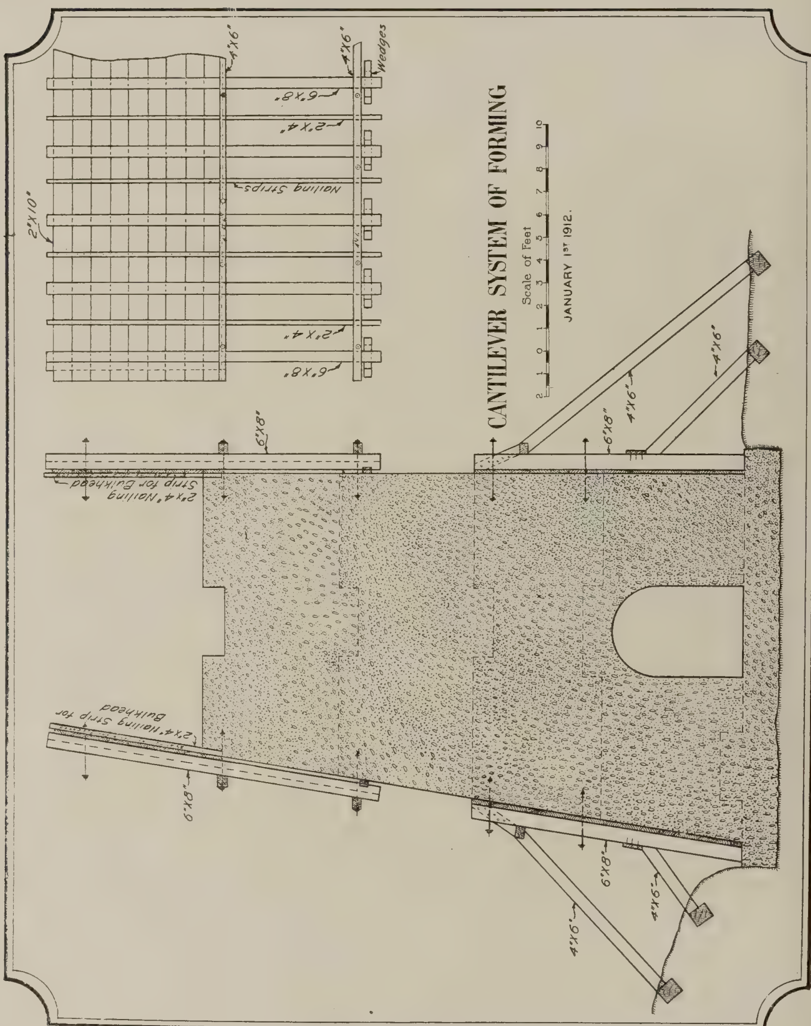

CANTILEVER SYSTEM OF FORMING
Scale of Feet
2 1 0 1 2 3 4 5 6 7 8 9 10
JANUARY 1ST 1912.
2"X10"
4"X6"
Wedges
6"X8"
2"X4"
Nailing Strips
2"X4" Nailing Strip for Bulkhead

Fig. 1.

taken up bodily with the derrick, and lifted to the next position, being allowed to rest on the top bolts while the wales are being bolted on; then the lower row of bolts can be removed and used repeatedly, with only the loss of one nut each time, which loss is inexpensive.

In the writer's opinion, this is the most economical system of forming in use to-day, both as to cost of material and cost of erection, for the lumber and other material can be used over and over repeatedly.

Fig. 2. Showing cantilever system of forms in use on lock wall.

This system can be used to advantage on all classes of heavy masonry, such as lock and dam work, bridge piers and retaining walls; especially so where cyclopean masonry is used, as there are no rods or braces to obstruct the laying of large stones or in handling concrete buckets, and these forms can be used at a height of 100 feet, or more, as easily at a height of 20 feet.

Recent Lower Mississippi Valley Waterway Improvements

BY

Maj. CLARKE S. SMITH*
Corps of Engineers

The recent works to which reference will be made are, in general, located in and along the Lower Mississippi from its confluence with Ohio River to the mouth of Red River and in and along the most important tributary systems below the mouth of the Ohio, viz: the Red, Yazoo, Arkansas, and St. Francis, these systems including the Ouachita and the White rivers, branches of the Red and Arkansas, respectively. The works of greater importance include revetment construction, dredging, levee erection, lock and dam building and snagging operations.

LOWER MISSISSIPPI RIVER.

Revetments. Some of the characteristics of the Lower Mississippi River are developed to the maximum degree between the mouths of the Ohio and Red rivers. Among them, the caving banks furnish the necessity and the rapid current increases the difficulty of revetment. Huge quantities of earth caving into the river cause the channel to deteriorate and supply the material for the bar formations. The current erodes the narrow necks accompanying the winding channel and threatens or creates cut-offs which disturb the river's regimen; it eats into the banks and menaces or destroys the levee embankments.

Critical situations are usually found (a) along some city front where the value of property makes it undesirable to move back the levee; (b) between some river, bayou, creek, or lake and the Mississippi, where a new levee could not be constructed without the diversion of the smaller watercourse or where a very long new levee would be required to encircle the lake or where a lake or bayou

*Formerly in charge of Third (Vicksburg) District Lower Mississippi River Improvement. Now in charge First and Second (Memphis) Districts.

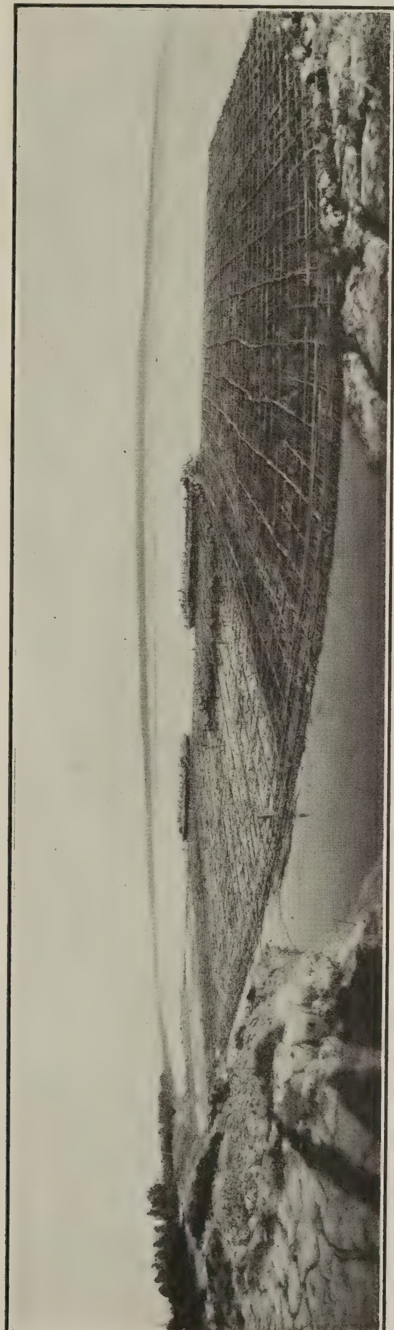


Fig. 1. Fascine channel mattress for bank protection on Lower Mississippi being ballasted just before sinking.

crossing would form a very unstable foundation for the new levee; (c) at a narrow neck where a cut-off would shorten the river and start additional destructive caving, and (d) at localities where it is desirable to prevent the removal of the channel from important locations. Because of the expensive nature of revetment and of the limited funds and facilities available, construction work of this kind is usually confined to these critical situations. Thus, as examples, important revetments have been placed at (a) Greenville, Miss. (478 L.)*, Helena, Ark. (307 R.)*, and Memphis, Tenn. (230 L.); (b) Walnut Bend (280 R.), Old Town Bend (325 R.), Panther Forest (452 R.), and Vaucluse (487 R.), Ark.; Fitlers (550 L.) and Albemarle Bend (568 L.), Miss.; (c) Ashbrook Neck, Ark. (471 R.), and (d) Hopefield, Ark. (229 R.) and Delta Point, La. (592 R.).

The work at Albemarle Bend, Miss. (568 L.), 30 miles upstream from Vicksburg, Miss., completed March 1 of this year, is a recent illustration of present-day revetment construction. The river had been caving in this bend along a distance of about 8 miles for years, the rate of advance into the bank averaging some 440 feet per annum. Several lines of levees had been constructed at different times and each in turn had caved into the ever-encroaching channel. The river was approaching Steeles Bayou and new levee loops, located at sufficient distance from the caving bank to have a satisfactory probable life as limited by that destructive force, would have cost an immense sum and no doubt would have required more than one season's work—an important consideration, since the river was about to cave into the existing levee. A revetment of 11,650 linear feet and a new levee, about 3 miles long—a short loop compared with new levee, which would have been necessary without a revetment—were begun in August, 1910, and both works had been completed at the end of about six and one-half months. The channel mats were 250 feet wide and two kinds were used, viz: the fascine and the framed types, over lengths of 11,000 and 1,550 feet, respectively, the object being to test the value of each under conditions as nearly similar as possible. The revetment was the most extensive ever constructed in the Lower Mississippi River in a season at one locality and, together with the new levee, cost above a half million dollars. Fig. 1 will give an idea of the

*Figures indicate miles below Cairo. R refers to right, and L to left bank of river.

appearance of a channel mat floating on the water over its final subaqueous position.

With the extension of operations in revetment work, an increase in plant has been necessary. Thus, the First and Second District now has sufficient vessels to carry on the work of three new revetments and the Third District is similarly provided. But additional plant is being constructed or considered with a view to providing fourth parties for these districts. The tendency in constructing new plant is to increase its durability at increased first cost. With very extensive plant the annual labor and cost of repairs is large, and as the number of vessels is increased it is desirable to make their construction such as to decrease the annual repairs on each.

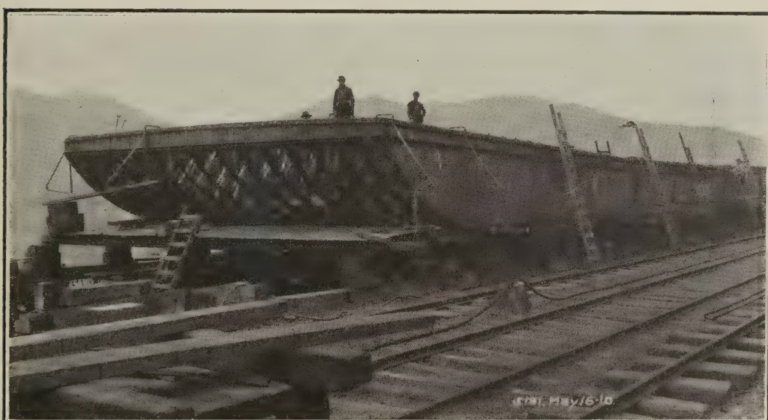


Fig. 3. New steel barge for use in channel improvement. Lower Mississippi River.

Untreated wood will last only about six or seven years in the valley of the Lower Mississippi, due to the alternate heat and dampness. New barges are consequently being constructed of creosoted lumber or of steel; quarterboat hulls of creosoted lumber and towboat hulls of steel. Fig. 2, the insert sheet, shows the design of steel barges of 500 tons' capacity now under contract for next year's delivery, and Fig. 3 is a photograph showing one of the steel barges received this year. The barges are used in transporting material for revetment construction.

The hydraulic grader recently completed for the Third District consists of a 3-stage centrifugal pump rated at 1,200 gallons per minute at 470 feet total head, operated by a direct-connected

36-inch 7-stage steam turbine developing 225 b.h.p. at 1,300 r. p. m. This unit is more satisfactory than the reciprocating units formerly in use because of greater economy in operation and repairs.

For repairing and painting the hulls of towboats, barges, quarterboats, etc., dry docks are necessary. A new floating dry dock capable of raising steamboats, towboats, hydraulic graders, etc., was completed and delivered the past spring for the Third District. It is steel throughout, the length and breadth of the hull being 156 and 56½ feet, respectively, and is operated by vertical steam engines direct-connected to 8-inch centrifugal pumps, one unit placed in the center of the side wells on each side. Fig. 4 shows this dry dock just before being launched.

A marine railway was recently completed opposite Memphis, Tenn., and is intended primarily to dock the dredging plant, but is also used in repairing other Government boats. It has a capacity for vessels up to 1,500 tons. Six steel cradles, operated by chains and sprocket wheels, move along the inclined track-ways of reinforced concrete. The structure rests on concrete piles to low water, below which it is entirely of timber.

Dredging. The dredging carried on annually in the Lower Mississippi River is prosecuted with a view to maintaining a channel below Cairo at least 9 feet deep and 250 feet wide throughout the year, except when the river is closed with ice. Nine dredging plants are now available, and as sand is the usual material encountered, the hydraulic dredge with the "dust-pan" suction head has been the type adhered to. This kind of dredge has produced good results as a general rule, but on rare occasions, as at Plum Point, Mo., last season (1910), clay is met with and unsatisfactory progress follows. This clay seems to be unusual elsewhere in the Lower Mississippi River dredging operations, and may be a local formation connected in some way with the earthquakes and the upheaval reported at that place in 1812. The amount of material dredged annually has varied during the past eleven years between 197,847 cubic yards in 1905 and 2,167,766 cubic yards in 1908.

Levees. It may be noted that the Lower Mississippi is similar to many rivers in receiving increments to its volume from important branches at not very great distances along its course. It differs in this respect from the lower Nile. The latter stream, below the Atbara, flows for more than 1,500 miles—a distance about 50 per cent greater than that from Cairo to the Gulf of Mexico—without a tributary of any consequence: as its waters advance, its

volume no doubt diminishes in size, due to evaporation, but the river's strength nevertheless enables it to traverse the desert and reach the sea.

The tributaries to the Mississippi are important for several reasons in connection with its improvement, one of which is that they make necessary the openings in the levee system, and these gaps, except that due to Red River, must remain in order to provide for the drainage.

There are, however, a number of minor streams, each of which

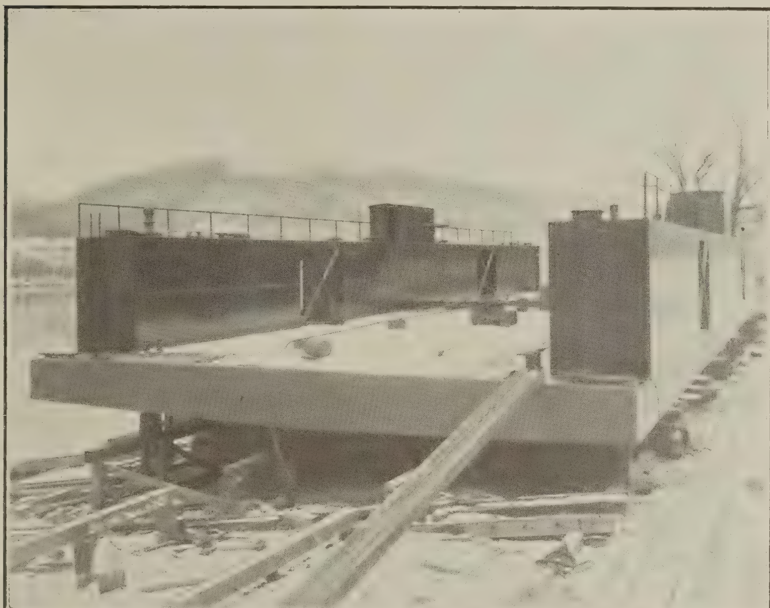


Fig. 4. New Steel Dry Dock for Third (Vicksburg) District. Lower Mississippi River.

may be diverted into one of the tributaries above referred to and some attention is now being, or very probably in future will be, given to them, particularly on account of the large land areas which may be reclaimed. Thus, a plan has been made to divert the waters of Cypress Creek, Ark., probably amounting at times to 2,100 cubic feet per second, through a canal along a number of bayous and lakes running generally parallel to the Mississippi and into the tributaries of the Red. The levees along the south side of the Arkansas may then be joined with the Mississippi system and

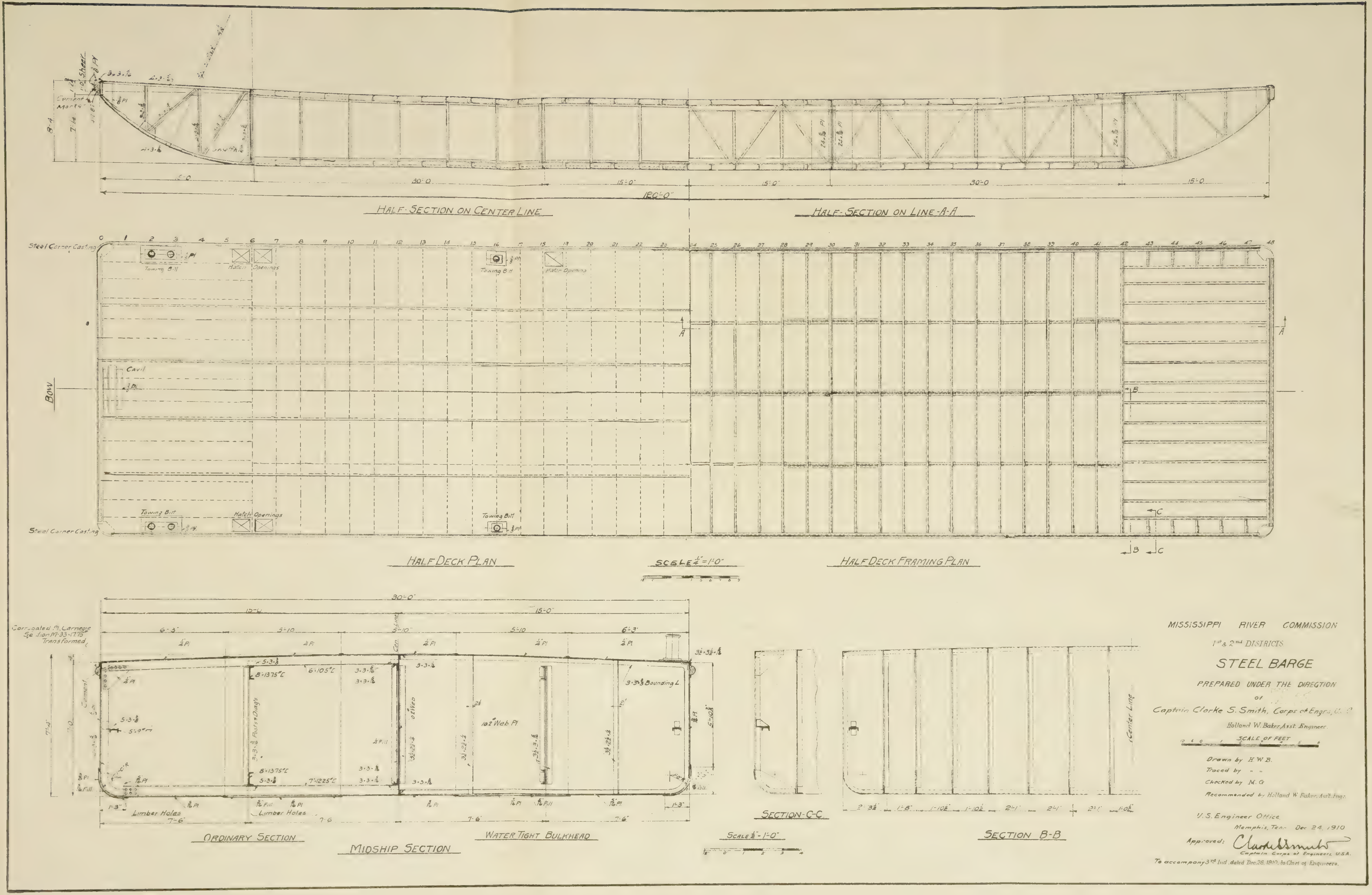
there will result a continuous levee line between Pine Bluff, Ark., and a point near the mouth of Red River, a distance of some 414 miles, thus forming a complete protection for the alluvial lands of Southern Arkansas and Northern Louisiana against the overflow of Arkansas and Mississippi rivers. The proposed canal will drain a number of other very flat water sheds in addition to that of Cypress Creek, the total acreage benefited being about 555,000, at an estimated cost of $1.14 per acre, a very small tax considering that the value of cotton and rice lands varies between $25 and $110 per acre. After the work is finished the Upper and Lower Tensas levee system will protect an area of about 5,000 square miles which was formerly subject to overflow.

It has been proposed to divert Red River through the Atchafalaya, to close the present gap in the levee system at the mouth of the former and to provide for navigation between it and Mississippi River by means of a lock. The completion of such works would result in the reclamation from overflow of a large area, and the benefits and disadvantages are at present under consideration.

Should these projects be carried out, the Upper and Lower Tensas levee systems would then be complete as far as relates to their length. At present the levee cross section and height at various places are deficient.

The Lower St. Francis Levee District may be extended somewhat at its downstream end, and its upstream end requires a small amount of similar work. The cross section and the height of its levees at various places should be increased.

The Upper St. Francis Levee District will require an addition of 23 miles of levee at its downstream end to bring it to the high ground near New Madrid, Mo. Before this work can be completed, however, it will be necessary to dispose of the water of James and St. Johns bayous in some manner similar to that proposed for Cypress Creek. The extension of this system to New Madrid will benefit a considerable area, but some of the largest levees in this Upper St. Francis District are involved and no work is planned on this extension at present. Near the upper end of this system the gap left for the egress of Big Creek is now being closed by a levee and the water will be siphoned over temporarily, and later diverted into interior bayous emptying into St. Francis River. A levee has just been completed across the lowlands between Cape Girardeau and Commerce Bluffs, Mo., and the extension above referred to, enlargement of existing levees, some new levees of small size


HALF-SECTION ON CENTER LINE
HALF-SECTION ON LINE-A-A
180'-0"
Steel Corner Casting
Towing Bitt
Hatch Openings
Cavil
Bow
HALF DECK PLAN
SCALE 1/4"=1'-0"
HALF DECK FRAMING PLAN
Corrugated Pl. Carnegie Section M-33-17.75# Transformed
Cement
Limber Holes
ORDINARY SECTION
MIDSHIP SECTION
WATER TIGHT BULKHEAD
SECTION C-C
SECTION B-B
Center Line
MISSISSIPPI RIVER COMMISSION
1st & 2nd DISTRICTS
STEEL BARGE
PREPARED UNDER THE DIRECTION
of
Captain Clarke S. Smith, Corps of Engrs. U.S.A.
Holland W. Baker, Asst. Engineer.
SCALE OF FEET
Drawn by H.W.B.
Traced by " "
Checked by M.G.
Recommended by Holland W. Baker, Asst. Engr.
U.S. Engineer Office
Memphis, Tenn. Dec. 24, 1910
Approved: Clarke S. Smith
Captain Corps of Engineers, U.S.A.
To accompany 3rd Ind. dated Dec. 28, 1910, to Chief of Engineers.

just below Commerce, and maintenance will constitute the future work on this system.

As far as length is concerned, the Reelfoot Levee System is complete, extending from the bluffs at Hickman, Ky., to the high ground at Slough Landing. Its cross section, however, requires enlargement at a number of places.

The Upper Yazoo Levee District is now entirely complete and only requires maintenance and protection from the caving banks. The Lower Yazoo requires further enlargement, maintenance, protection from caving banks and a downstream extension. The successful diversion of the Yazoo River through Lake Centennial and along the Vicksburg city front has resulted in the silting up of the former mouth of that stream, a condition favorable to the future extension toward Vicksburg of the Lower Yazoo system across the former channel of the Yazoo. An extension of 18 miles will lower the back-water flood plane about 5 or 6 feet at high Mississippi River stages and will thus benefit a considerable territory.

Projects may be undertaken in future with a view to obtaining protection by levees for some of the comparatively narrow districts between the left bank of the Mississippi and the bluffs.

The following table indicates the growth of the entire lower Mississippi River levee system during the past ten years:

Name of District.	Contents of levees in cubic yards in 1901.	Contents of levees in cubic yards in 1911.	Per cent increase
Upper St. Francis	287,198	3,448,722	1100
Lower St. Francis	13,270,905	26,016,396	96
Reelfoot	140,311	1,713,458	1123
White River	7,390,730	11,346,107	53
Upper Yazoo	19,190,503	30,634,397	59
Lower Yazoo	30,664,029	44,703,267	45
Upper Tensas	29,194,708	39,321,789	34
Lower Tensas	16,683,503	24,647,388	47
Atchafalaya	18,596,594	22,362,015	20
LaFourche	7,624,410	9,772,443	28
Barataria	2,864,222	3,715,665	30
Pontchartrain	13,893,720	18,942,550	36
Lake Borgne	3,225,835	4,416,321	37
Totals	163,026,668	241,040,018	48

During the past decade the increase in volume in the controlling levees—the contents of abandoned parts having been deducted—amounted to 78,013,350 cubic yards. The total length of the levee system is now 1,500 miles, and the total area protected about 26,569 square miles. Interesting figures for comparison are the contents of the excavation in the Central Division of the Panama Canal, which includes the Culebra Cut and amounts to 117,972,018 cubic yards, and the total in canal of 212,445,766 cubic yards; the length of the Great Wall of China, about 1,500 miles, and the combined area of Massachusetts, Connecticut, Rhode Island, Maryland and Delaware, which is 25,758 square miles.

TRIBUTARIES.

The tributaries of the Mississippi below the mouth of Ohio River furnish long systems of waterways. The most important, as already mentioned, are the Red, Yazoo, Arkansas and St. Francis river systems.

Red. The Red River System (including the Black and Ouachita*) with its branches furnishes a waterway under maintenance and improvement of about 1,800 miles in length, although a considerable part of this distance is navigable only at the medium stages. Red River itself is a stream which is constantly changing by reason of the erosion of its banks. It flows through a light, fertile, alluvial red soil, of which its strong current readily carries a considerable amount as sediment. The improvement has consisted in removal of snags, drift, rafts, construction of levees, closure of outlets, the destruction of some protruding reefs and the construction of dikes and revetment. The great raft extended over a distance of 92 miles, the upstream end being about 27 miles above Shreveport, La. Its removal was followed at a later period by the closure of outlets and the construction of levees. The results are very remarkable. The river bed has enlarged and the survey now in progress, when compared with that of 1889-1892, shows that over a distance of about 55 miles in the vicinity of the great raft the river bed has deepened an amount varying up to as much as 25 feet. An enlarged and deeper section of the river and the increased volume of water, due to its confinement in one channel, has benefited navigation. In order to remove obstructions due

*The Ouachita flows into the Black River at Trinity, La.; 56 miles farther on the Black flows into the Red River. Thus the improved channel from the Red River to Camden, Ark., is called the Ouachita and Black River.

to bars at low stages, a new dredge—the *Waterway*—has been designed and is now being constructed. This dredge is to be of the self-propelling hydraulic type, with 16-inch centrifugal pump and revolving cutter suction head. It is to have a steel hull 142 feet by 34 feet, and is to be supplied with condensing engines and also electric light and refrigerating plants. The necessary steel pontons and pipe line will accompany the dredge. As its work will be accomplished during low stages of water, the calculated draft of the dredge has not been allowed to exceed 3 feet.

Ouachita and Black River is the principal tributary to Red, from which, however, it differs greatly in having stable banks, and

Fig. 5. Lock and Dam No. 8. Ouachita River, Franklin Shoals, Ark.

consequently a definitely defined course and little sediment. Its improvement consists in the canalization now in process of construction and in snagging. Eight locks and dams are projected and four are now under construction in various stages of progress. This canalization will provide an all-the-year navigation of $6\frac{1}{2}$-foot least depth for a distance of 287 miles upstream from the lower lock, the latter being 72.6 miles from the confluence of Black and Red rivers. The dimensions of the locks adopted are 300 feet between hollow quoins and 55 feet clear width. Lifts vary between 5.5 feet and 14.8 feet, and the dams are of the movable

needle type provided with navigable pass, weir, and drift pass. The foundations range from hard blue clay to light sand, necessitating the use of piling. Cofferdams constructed of round piling, waling and single lap sheet piling and filled with puddle are used to withstand stages of from 10 feet to 14 feet, but floods occur to such an extent as to limit the working seasons between three and six months per year, and higher cofferdams would add but little to that time. Lock and dam No. 8, begun in 1908 and carried on by force account, is the most advanced in progress of construction. The accompanying illustration, Fig. 5, was taken at the close of the past working season and shows the work at lock and dam No. 8 at that time.

Recently, a steel hull combination snag and dredge boat was completed for Ouachita River. It is provided with an orange-peel bucket, removable when the boat is required for snagging.

Yazoo. The Yazoo System consists in navigable waterways aggregating some 800 miles in length, located in the flat delta lands of Mississippi. The banks are all stable, except for sliding, and the improvements have consisted in snagging, dredging, closure of outlets and, as previously referred to, diversion of the mouth of the river to provide a harbor for Vicksburg and an entrance navigable at all stages into the Mississippi River. Excellent results have been produced by this recent diversion of the mouth of the Yazoo through the lake formed by the cut-off of 1876 in Mississippi River opposite Vicksburg. Yazoo River now enters Mississippi River at a concave bend 9 miles farther downstream than formerly. A harbor for Vicksburg, thus produced by the diversion, forms the communication between the two rivers without the obstruction of the bar which formerly existed at the mouth of the Yazoo. The Upper and Lower Yazoo levee system now prevents the flood water from the Mississippi from traversing the Yazoo, except as back water near the mouth of the latter. Recently a combination snag-and-dredge boat similar to the one referred to for the Ouachita was completed for the Yazoo System.

Arkansas. Arkansas River, including the White and its branches, furnishes navigable watercourses under improvement over 1,300 miles in length. The work done has consisted principally in the construction of regulation works such as revetment and dikes, in dredging, snagging, and, on the lower part, in levee construction. The Arkansas resembles Red River in its caving banks and changing course. Its maximum volume is more than 250 times

its minimum. Congress recently provided funds for two dredges for the Arkansas, and plans for their construction are now in progress.

White River enters the Arkansas just before the latter reaches the Mississippi. Its banks are generally stable, except in its lower course, and it carries but little sediment. A project was adopted for the construction of ten dams and locks with at least 4 feet of water on the miter sills to provide slackwater navigation from Batesville, Ark., to Buffalo Shoals, 89 miles farther upstream. Three of these locks and dams have been completed, the locks being 175 feet long between hollow quoins, 36 feet in interior width with lifts of 14 and 15 feet. The dams are the rock-filled timber crib type without any movable parts. The structures are founded upon rock and on gravel, piling being used in part of the work.

St. Francis. This system adds more than 300 miles of navigation to the Lower Mississippi waterways. The improvements have consisted principally in snagging. It is now protected from the overflow waters of the Mississippi by the Upper and Lower St. Francis levee systems and, as it traverses flat delta lands, its banks are stable and it carries but little sediment.

SUMMARY.

The Lower Mississippi River, with the tributaries mentioned, comprises a system of waterways aggregating more than 5,000 miles in length. The improvement works now in progress consist principally in placing revetments, levee building, dredging, and snagging in connection with the Mississippi and similar operations accompanied by lock and dam construction in and along the tributaries. These works already have, and in future will, facilitate navigation and have benefited more than 26,000 square miles of fertile alluvial territory.

Development and Tactics of the Military Bridge Equipage

BY

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From the beginning of military history, the records of wars contain numerous descriptions of the passage of rivers and other natural obstacles by military forces. Such passages were effected in early times by improvised bridges, constructed of such materials as might be at hand. The long and dangerous delays resulting from such methods led to the design and construction of bridge equipages that could accompany an army on the march and render it independent of the resources of the field of operations.

About 766 B. C., Semiramis*, on her expedition into India, constructed a large number of light boats for ferriage purposes, which were carried on pack animals. These boats were sectional, and could be combined to form large floats by means of iron attachments.

In 326 B. C., Alexander the Great, at the opening of a campaign which involved the passage of the Indus, prepared in advance an equipage consisting of light sectional boats of two and three parts. He also constructed an equipage of boats made of the skins of animals for the anticipated passage of the Oxus. Hannibal made use of a similar equipage for the passage of the Rhone.

Julius Cæsar was the first to provide an army with a completely organized bridge equipage, including the roadway. The boats of this equipage were of willow frames covered with skins.

In the fourth century of the Christian era, the Emperor Julian, in his expedition against the Persians, formed a similar bridge equipage, which was successfully used at the passages of the Tigris, the Euphrates, and the Halys. After the defeat of Julian, the Persians promptly copied this equipage and used it for the passage of the Tigris.

*The History of the European Bridge Equipage.—Birago.

During the decline of the Roman Empire and the prevalence of the feudal system, military art of all kinds was at a low ebb and portable bridge equipages were unknown. In the Fifteenth Century the growth of powerful monarchies, following the decline of the feudal system, led again to the organization of well equipped armies.

The wars of the Sixteenth Century in the low countries show many instances of bridges constructed of boats and rafts, e. g., at the sieges of Maestricht, Antwerp, and Venloo. These were all, however, improvised bridges, but the Thirty Years' War (1618 to 1648) showed that a lesson had been learned, and the armies are found equipped with portable bridge equipages suitable for the transportation of the heavy artillery carriages of that time. The supports were boats, 28 to 35 feet long, generally of oak, weighing from 4,300 to 4,900 pounds. The flooring of these bridges was similar to that of to-day, consisting of pine balk and oak chess. The roadway was from 12 to 13 feet, over all, in width. This equipage was used by Brunswick, Tilly, and others in the passage of the Weser, the Main, the Elbe, the Vistula, the Rhine, and the Danube. At this epoch, military operations consisted almost entirely in taking up skillfully selected positions for battle, or in the careful conduct of sieges. Time was an element of small importance, and almost all operations were slow. The enormously heavy equipage just described was a very small detriment to the movement of armies, and further, the artillery carriages of that day were practically as heavy as the ponton wagons.

As rapidity of movement became recognized as a governing feature in warfare, efforts were made to lighten the loads of military trains. These efforts were at first devoted almost entirely to the artillery, and the ponton trains could hardly keep in touch with the rear guard. Long delays resulted whenever streams were to be crossed, with sometimes fatal results, as in 1705, when a day's delay in the arrival of the bridge train prevented the junction of the Sardinian army with that of Eugene of Savoy, giving the French forces time to effect a concentration on the Adda that nullified an otherwise successful campaign.

Prior to the incident just referred to, the Dutch had constructed (in 1672) a much lighter train. The boats were of iron, 18 feet long, and weighed only 690 pounds. The wagons of this train carried the materials for one complete bay and required only six horses for each wagon, whereas those of the Thirty Years' War

required from fourteen to sixteen horses for a load of a boat alone.

The example of the Dutch was rapidly followed by all the nations of Europe, and light boats appeared on every side. Those of the Spanish, French, and Portuguese were of copper; of the Dutch, the Saxons, the Prussians, and the English of iron; of the Russians of canvas on frames. Most of the powers retained, however, their wooden equipages considerably lightened but still slow of movement. The light trains moved with the advance and the heavy trains with the rear guard.

The effort to decrease the weight of the light trains was, like most military fads, overdone, and stability and safety were sacrificed. A reaction very shortly set in, inaugurated by the French, and the light boats were increased in size and weight so as to have a supporting power of about 5½ tons. This increase was found insufficient in the first Silesian War (1742) and in the wars of the same time in Northern Italy. In fact, it was found impracticable to bridge with the metal boats such rivers as the Rhine and the Danube, the Po and the Ticino.

Metal as a material began to fall into disfavor; the iron rusted, all kinds were easily punctured, wore out quickly in service and were difficult of repair. The devotees of the metal boat exhausted themselves in ingenious devices to correct these faults, but none were successful. About 1770, the French adopted a new bridge train, designed by General Gribeauval, with pontons of oak, 35 feet long, weighing 4,065 pounds. The supporting power of this ponton was 18½ tons. This train was used in the early wars of the French revolutionary government and in Napoleon's German campaign of 1805. It proved too heavy for satisfactory transportation and was abandoned. Austria, about this time, adopted a ponton 26 feet long, weighing 1,540 pounds, which was copied by Bavaria, Wurtemburg and Baden. Saxony, in 1789, adopted a wooden ponton 22 feet long. Napoleon, during the Italian campaign of 1796, had constructed at Alessandria a train of forty pontons on the Austrian model. These appear to have been satisfactory to him, as he later had constructed in Dantzig a train of one hundred pontons on the same model for his Russian campaign of 1812. The fate of this train was a sad one. The wretched roads of Russia led to the abandonment, at Dwina on the advance, of all but twenty-eight of the pontons, and these were burned on the retreat for lack of transportation, even though it was known

that some means would have to be provided for the passage of the Beresina.

The close of the Napoleonic wars found no nation with a satisfactory bridge equipage. All had adopted wooden pontons except England and Saxony, which held to the iron; and Russia which retained the canvas. There followed an epoch of absurd experiments: so-called "rolling bridges," in which the wagons were to be used as points of support, trestle bridges, rope bridges, suspension bridges, framed truss bridges were suggested, and in some cases used with success at peace maneuvers. Of these, the trestle bridge alone developed qualities that justified its incorporation in the military bridge equipage.

In 1824 a new system appeared, designed by Lieut. Col. Colleton of the English staff. The supports consisted of hollow wooden cylinders, 14 feet long, used in pairs. The supporting power of each pair was 8.9 tons. The total weight of a wagon loaded with materials for a complete bay of 22 feet was only 4,000 pounds. Peace trials at Chatham were very favorable to this equipage, but it was soon abandoned, as the pontons were absolutely unsuitable as individual boats and developed minor defects.

From 1824 to 1845, most of the nations of Europe experimented with bridge equipage, and the struggle for lightness of wagon load with ample supporting power led to a considerable adoption of the sectional ponton. The introduction of the trestle into bridge equipage was common, and a special trestle equipage was furnished the French sappers for the passage of ditches of fortified places.

The use of wood for pontons was general, though the English and Danish retained the metal boats, and the Russians the canvas. Several countries adopted equipages of two weights (the French three) for use under different conditions. The Piedmontese, Prussians and Saxons inaugurated the scheme of arranging the same equipage for bridges of different widths and supporting power for the use of the different arms. Too great complication to attain this object diminished the serviceability of some of these equipages. Many of the pontons of this period were of such poor design as boats that they could not be maneuvered for the casting of their own anchors, and small boats for the purpose were included in the bridge equipage.

The first bridge equipage in the United States was organized during the Mexican War. Two complete trains of inflated India rubber pontons were constructed and sent into the field. They

proved very unsatisfactory, owing to the perishable nature of the material, the lack of weight (permitting great oscillations of the bridge), and the ease of destruction by a single rifle bullet. This system was abandoned in 1858, and the development of a new one begun. Careful study of the European pontons led to the selection of the French, Austrian, and Russian models for trial. Types of these boats were constructed and thoroughly tested under heavy loads, storms, tides, and floating ice. Corrugated iron was tried for the boats, but was found too weak, unless the boat was made unjustifiably heavy; it was easily injured in the water and in transportation and difficult of repair. Wood (white pine sheathing and oak frame) was selected for the material of the pontons. The need in our country of a bridge train of the most substantial character, suitable for wide and rapid rivers and the heaviest trains, necessitated the adoption of two types of bridge equipage, in order that a light, easily moving train might be available for the most rapid cavalry movements.

The boat for the heavy train was patterned on the French wooden ponton, and for the lighter train on the Russian canvas covered boat. To each train were added a few trestles of Birago type, generally adopted in Europe. This equipage has undergone practically no change since 1862, and is the one still in our service. The heavy equipage is called the reserve equipage, and the light the advance-guard equipage.

At present most foreign nations favor a steel boat, though the wooden boat is retained in some equipages. In Germany a territorial development has taken place, providing a very heavy train for the Rhine district, and lighter trains with some variations in the states on other frontiers. In many countries the effort to obtain an elastic equipage is apparent. Sectional boats of three sections have been adopted, using a single section for support on a foot bridge, two sections for a cavalry bridge, or three for a wagon bridge; the lengths of the lighter bridges being correspondingly increased. These sections are frequently square-ended, making poor individual boats; but in Austria sectional boats have been adopted, all sections pointed bow and stern, overlapping a few feet when assembled. The Japanese use a sectional boat in two halves, each half weighing only 500 pounds and hauled by a single pony. This is the lightest equipage now in use, unless the lance boats of the German cavalry are to be considered as bridge equipage.

For the heavy train, the American wooden ponton probably has

no superior. It is fairly light, has good supporting power, will stand without injury severe twists in transportation, is easily and quickly repaired, and is a good boat for ferriage and general rowing. The remainder of the equipage of the heavy train would also be hard to improve upon. It is strong, readily proportioned to handling by the pontoniers, and produces a strong, stiff bridge. Any radical changes in this equipage should be very carefully considered before being adopted.

The advance guard train is, on the contrary, not entirely satisfactory for the duties demanded of it. The wagon loads are too heavy, the pontons are poor boats for ferriage and the bridge is inelastic. A better type would be one following the Japanese or the Austrian sectional boat, permitting several lengths and widths of bridge with the same equipage, arranged for light loads on two-horse wagons.

The trestle is still a component of most foreign bridge equipages, as well as our own. Some are similiar to our (Birago) trestle, others have four legs; the Belgians use six legs. In Holland, provision is made to set the trestle up in the boat, attaching it with pins to cleats in the bottom. This permits a higher roadway for narrow streams with high banks, or for a gradual slope from a high bank on a wide stream. The Birago trestle is still carried in the United States bridge trains. Other types have been suggested and tested in recent years. One designed by Major Rees, with some modifications, is believed to be an improvement on the Birago in that the legs are vertical, permitting greater ease of adjustment.

The advisability of carrying trestles with a bridge train is more than doubtful; most officers who have served with our trains in the field regard the trestle as a bugbear. Its operation is always troublesome; on streams with fluctuating levels the constant raising and lowering of the roadway is tedious and difficult, and interrupts the use of the bridge. On soft or uneven bottom, the footing is insecure. If several trestles are used the lack of longitudinal stiffness becomes a great menace, the stability of the abutment sill being the only safeguard. At Fort Riley, in 1902, it was necessary, in order to secure longitudinal strength, to use four ropes as a diagonal bracing throughout the length of the trestling. To counteract contraction and expansion in these ropes it was found essential to introduce a rackstick in each member (four to each bay). These had to be slackened each night and tightened each morning, as well as after the passage of any column. Another fault of the

trestle is the difficulty in clearing it of accumulated drift, which can generally be pushed down to pass under a boat. The lack of yielding in a trestle to the impact of drift is still another defect. With a bridge, at one of the maneuvers, it was found necessary to cast an anchor above each trestle and run a cable to the trestle cap to provide a gentle incline for the drift to bring up on. Before this was done, one trestle was broken by the impact of floating logs.

In the United States Ponton Manual, the advantageous uses quoted for trestles are: 1st. Over dry ravines, marshes, and streams too shallow to float a ponton. In any of these cases boats can be moved by hand and placed in position by the pontoniers more rapidly than trestles can be carried out and assembled. 2d. On shallow tidal streams where the pontons would ground at low water on an uneven bottom. If the range of tide is large (more than 5 feet) it is essential to use boats, allowing them to ground to prevent too great inclination of the shore bay for the passage of loaded wagons. If the foreshore is uneven, it can be easily dressed at low water under the pontons or they can be blocked up. If it consists of sharp boulders, protection can be given the pontons with planks, bales of hay or brush. If the sheathing is broken it can be repaired, but it is not likely to break under the dead load of the bridge alone.

The principal objection to the trestle is, however, one of transportation. It is doubtful if any military movement has ever been conducted where the demand for transportation was not greater than the supply. Scarcity of transportation is likely to cripple most severely the engineer trains, as the trains for ammunition, subsistence, and medical supplies will always be served first. In a ponton bridge train boats are the essential element; everything else can be more easily improvised. The more boats the better for any crossing, enabling rapid work on wide streams and a multiplication of bridges on small ones. For short bridges across dry ditches, or ramps for crossing railroad embankments, the trestle is useful, but supports for such purposes can easily be made by using two or more boats superposed on skids. In short, a trestle can be made of boats, but a boat can not be made of trestles. The trestle should be eliminated from our trains, substituting an additional boat wagon for each trestle wagon. I believe that with an intelligent train sergeant, the trestles would rapidly eliminate themselves in the field in active operations.

The tactics of the military bridge equipage is not extensive in

scope, and what there is of it is self-evident to those who have served with bridge trains in the field. It is unfortunate that some provision is not made for disseminating it to the army at large. There is practically nothing on the subject in the Field Service Regulations or the Engineer Field Manual, and very little in the text books taught at West Point. The only way, apparently, for line officers to acquire knowledge of these tactics is by experience, paid for by criticism by the engineers at peace maneuvers, and by the blood of pontoniers in the operations of war.

The insane attempt to bridge the Rappahannock at Fredericksburg in the face of the fire of the Confederate sharpshooters is an example of this lack of knowledge. The Count of Paris describes the attempt as follows:

> Before daylight a thin fog spread like a curtain between the two belligerents, but it was not dense enough to completely intercept the view from one side of the river to the other, and prevent Barksdale's soldiers from firing upon the Federal pontoniers who were bringing their boats into position one by one and adjusting the planks of the flooring. The firing presently became so brisk, that, despite their coolness, the latter were obliged to suspend their work. * * * Day broke, and the morning advanced without any progress having been made by the Federals in their work; their operations had been resumed three or four times, but the precision of the enemy's fire had always stopped them. The unfinished bridge was covered with blood; it was therefore time to bring matters to a crisis. * * * Hunt proposed to Burnside to ship some soldiers in the boats that had not yet been fastened together, and send them to the other side of the river to dislodge the enemy's sharpshooters; this is what should have been done at the very first. The Seventh Michigan, the Tenth and Twentieth Massachusetts, one thousand men in all, thus crossed rapidly over, losing but a few men. * * * Towards 4 o'clock the two bridges were at last completed.

Another example occurred in the preparations for one of the maneuvers at Fort Riley. An engineer officer being called on by the officer preparing the problems to suggest a maneuver involving the bridge train, seized the opportunity for instruction by outlining the forced passage of the Kaw River against an enemy's force holding the south bank. It being assumed impracticable to build a bridge under the enemy's fire, the maneuver suggested was to form a portion of the bridge train at a point on the river ready for construction, and at the same time to embark in ponton boats on a protected tributary stream a small force of infantry to

be ferried under cover of darkness to the south bank, drive in the pickets and sharpshooters and effect a lodgment to cover the construction of the bridge. The officer preparing the problems approved the maneuver, but, in drawing up the general situation, he considered it necessary to explain this movement by stating that the attacking force was not provided with sufficient bridge material. Apparently no other reason could be assigned for the failure to attempt to build the bridge under fire. Upon representations by the engineer officer, the clause in question was stricken out of the situation, the maneuver was splendidly executed by all troops engaged and, it is hoped, proved instructive.

The above examples are not quoted in a spirit of criticism, but merely to show the difficulties we may expect to meet in the future in service with bridge trains, due to the unfamiliarity of our commanders with their tactics. The following salient points of these tactics, while not covering the entire field, would be of value to us if tactfully imparted to those we serve and understood and remembered by them.

When a stream is to be crossed the most suitable point, other things being equal, is that where the stream is narrowest, enabling speed of construction, and, if the train is large, a multiplication of bridges. Next in importance, a point with low banks and easy approach should be selected. With high steep banks, the approaches are likely to require several times the work of the bridge itself.

If the enemy is opposing the passage on the opposite bank, tactical conditions are paramount. A point should be selected where the approach to the bank is concealed by trees or otherwise, and, if possible, with little cover on the opposite bank. The bank of departure should be the higher of the two, and should be the concave bank to permit a concentration of fire from a large arc on the enemy's position. The bends of alluvial streams are likely to offer a high steep bank on the concave side with a wide open sand or gravel bar on the opposite bank. Such a situation fills the above requirements and has the advantage that nearly all the work on approaches is required on the bank of departure, and can be executed by working parties while the bridge is being constructed.

If the further bank is occupied by the enemy he should, of course, be driven out by artillery or small-arm fire, if possible, before construction is begun on the bridge. A Spanish engineer

officer, writing recently, has stated that it is hopeless to attempt bridging a stream until the enemy's small-arm fire has been driven back a mile from the site and his artillery 3 miles. The experimental firing at Fort Riley, in the fall of 1909, against targets representing small bodies of troops in close formation, indicates that modern heavy field artillery would have to be kept back 4 miles at least. Our artillery must be relied upon to drive the enemy's artillery to these distances. Then, if the small-arm fire is not also silenced, an attack must be made before bridge construction is begun. This should be delivered in the ponton boats by infantry, the boats being rowed by pontoniers. A sheltered spot should be selected to launch and load the boats, but if none is available these operations can be effected with fair rapidity on the open bank. On a narrow stream, a vigorous attack of this nature should be successful, as it was at Fredericksburg. If it can not be made to succeed, it is certain that a bridge can not be constructed.

The methods of construction specified for use with our equipage, while pertaining more nearly to the domain of drill regulations than to that of tactics, deserve a few words here. The method by successive pontons will nearly always be the best. The methods by parts and by rafts are academic schemes that would be almost certain to cause delay, due to the difficulty in handling the boats and anchors. In very long bridges (forty to one hundred bays) speed can be much better attained, than by the above methods, by building rough rafts of two boats with all materials for two bays. As fast as needed four men can easily tow each of these out on the downstream side of the bridge to its head, dismantle it, piling the flooring material on the roadway and mooring the pontons ready at hand for the chess, balk, side-rail and anchor detachments. The method by conversion is still more delicate of operation and uncertain in results. It is likely to result in a complete wreck of the bridge on a stream of even moderate current. The only successful operation of this method in war, that I have been able to find, occurred in Napoleon's campaign in Germany in 1809. Just before the battle of Wagram, a bridge 177 yards long was thrown across an arm of the Danube by conversion. The conversion was effected in four minutes and men were crossing before the bridge was fairly in place. The success in this instance is probably responsible for the tenacious hold that this method has on ponton manuals. Situations are conceivable where this method might be worth risking, but it should be regarded as a forlorn hope, to be resorted to only when nothing else is possible.

Handling Our Ponton Equipage

BY

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Corps of Engineers

The handling of our reserve ponton equipment presents no great engineering problems. There are, however, on account of its weight and bulk, a great many difficult features connected with its use and transportation. Having for the past two years been on duty with a company which throughout that time has had for its special work the handling of this equipment, the writer of this article believes that some of his experiences may be of interest to other officers of the Corps.

Company L has been the ponton company of the Third Battalion since 1908. During that time portions of the heavy train have been carried to Fort Riley both by rail and also with the company on the road to the maneuvers in 1908 and 1910. A small portion of the equipment was taken to a military tournament at St. Joseph, Mo., in 1908, by rail. In 1909, a portion was taken by rail to a military tournament at Toledo, Ohio. At least a division was taken to Chicago in July, 1910. The order directing the movement of the Third Battalion of Engineers to San Antonio in March of this year called for two divisions of heavy and two divisions of light equipage. This entire equipment was handled by L Company.

In so far as the handling of this equipment in the field is concerned, there is little of interest that can be written. The arrangement heretofore followed out in this battalion, whereby the battalion quartermaster had control of the mules when in garrison and was in actual charge of the train on the march, has been found unsatisfactory. The reason is no doubt obvious. The commanding officer of the ponton company should have full and complete control of both teams and teamsters. He is, then, in a position to train both, and should be responsible for such training. And it is

not to be supposed that a teamster who is skilled sufficiently to drive a six-line team along the road, to halt, to march, to turn corners, this and no more, is a suitable driver for a ponton train. Such is far from the actual requirements of the case. Experience at the maneuvers has shown that valuable time may be gained or lost, depending upon the degree of skill of the driver. This is especially true when it is a question of loading and unloading the boats.

It has been found that the heavy train can march at about the same rate as infantry for long distances, and can make a longer day's march. Here again the skill of the driver comes into prominence. An unskilled driver will let the swing and wheel teams do all the work, instead of getting the full six "mule power" as he should. This is a point that needs constant attention. For short stretches, where it is necessary to cover ground at a more rapid gait, the train can move at a trot, but the walk should by all means be considered the normal gait. The harness as now used gives a great deal of trouble, some part about it is continually going wrong, getting loose or tangled, or in some way necessitating a halt for repairs or readjustments. The present harness, it is believed, should be changed, and something adopted more on the order of the artillery harness for six horses. A set of harness exactly like that used by the artillery would probably not be suitable, but something approaching nearer to that type would be more satisfactory than our present type.

Every one, no doubt, is familiar with the different methods of loading the boats on the wagons. The quickest and easiest way has been found to be as follows: The wagon is brought up to within about 12 to 15 feet from the boat, its axis in prolongation of the axis of the boat, tongue cut to one side, a cable is fastened to the mooring posts by means of a loop so that the strain will fall equally on both, it is then passed over the wagon and seized by from sixteen to twenty-four men, the balk are held down by a piece of pipe or timber passed over them near the front of the wagon and temporarily lashed. A strong "home made" roller, dimensions and construction as shown in Plate III, is held in place by two men about 2 or 3 feet in front of the boat; about four men stand by on each side of the boat to guide it. At a given signal the men heave on the rope, and if those detailed to do the guiding are spry enough the boat goes into place without a hitch. The iron dogs on the side of the wagons near the forward end frequently give

trouble. They are sharp pointed and stand almost vertical. Unless the boat is guided exactly into place it will catch on one or the other of these dogs, sometimes ripping a hole in the bottom. These dogs should either have a rounded knob on top, or should be bent considerably back from the vertical. The operation of unloading is almost the reverse; the roller can be omitted, provided the wagon can be brought near enough to the water.

In fact, in the field the problem is to handle objects of considerable weight and bulk with great speed. As there are always a sufficient number of men available, ''main strength and awkwardness'' is the rule in loading, unloading, and launching; but even so, with practice, ''main strength and awkwardness'' may be applied with considerable skill.

In loading the train on cars for transportation by rail, where sufficient flat cars are available, the problem is not so difficult, nor does it require more than a fraction of the time one would judge upon first thought. When possible, the flats are backed on a spur, the rear car being run as near as possible to the end of the spur. All the brakes are then removed or loosened and turned to one side. Wooden ramps 2 inches thick, 1½ to 2 feet wide, of sufficient length to bridge the gap between cars, and beveled at each end, are placed, two over each gap, so that wagons can be run from rear to front of train. A ramp is then set up against the end of the rear car as follows: A trestle, constructed specially for this purpose, as shown in Plate I, is set up against the end of the car and lashed to it by rings in the end of the trestle cap, five balk are then placed, one end on trestle cap, and the other resting on the ground, and supported near the center by a wooden horse. Chess are then laid in the usual way and held down by side rails. The wagons are then brought up near the ramp. A rope is made fast to the running gear, passed through a snatch block fastened at the forward end of the car, and hooked on to a team of two mules; three or four men guide the tongue as the wagon is hauled up the ramp. Once on the car, a squad of eight or ten men rolls the wagon to the front of the train where it is turned over to another squad to be blocked down and lashed securely. A short piece of 4 by 4 is securely spiked to the car floor on the inside and outside of each wheel, a 2 by 4 is then passed between the spokes of each pair of wheels and nailed to each piece of 4 by 4. In addition, each wheel is chocked by pieces of 4 by 4 nailed to the floor. (Plate II.) Each wagon is lashed by rope fastened to the

mooring posts at each end, and passing through the standard sleeves on the sides of the car and the lashing rings on the boats. This method has been invariably used and has always proven satisfactory. The wagon tongues are, of course, removed and securely fastened to the car floor underneath.

Loading this way is very quickly done and everything is on wheels, nothing is taken apart to be set up again when the des-

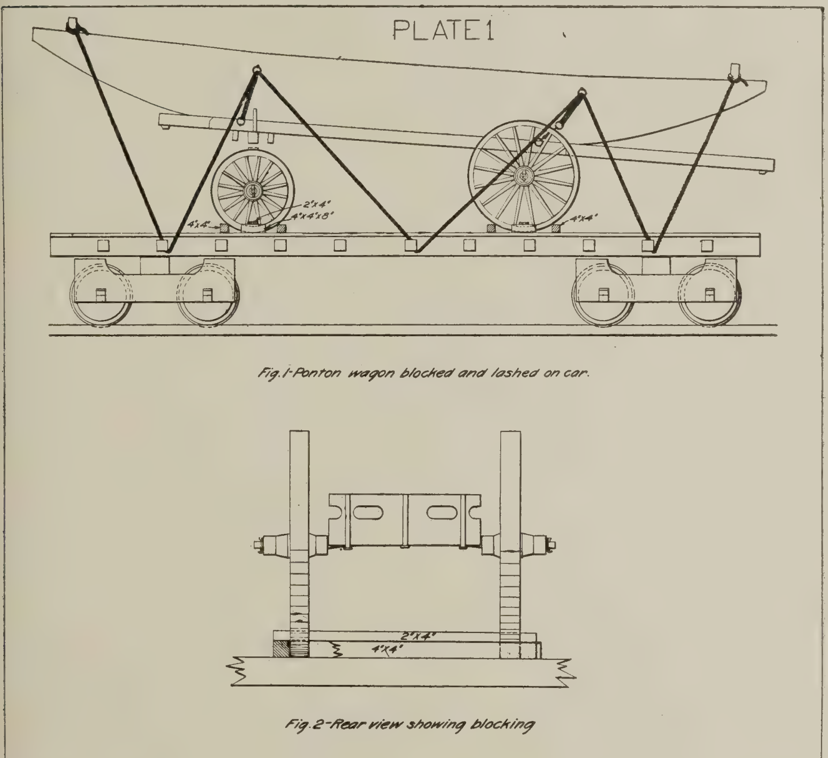


Plate I. Showing method of fastening pontons to car, when shipping equipage standing up; also method of chocking wheels while in transit.

nation is reached. The wagons are run off the train in a similar manner, and are ready for immediate use. Nor is this method so profligate in the number of cars used, for it must be remembered that a boat alone will take the space that would otherwise be occupied by both boat and wagon. A car 40 feet in length will accommodate a ponton wagon and a chess, or tool or forge wagon. To do this the chess wagon must be first loaded and the ponton run forward until its bow overhangs the rear of the chess; the running

gear of both wagons are reversed. A trestle wagon requires the same space as a ponton. Thus it will be seen that the minimum number of cars to accommodate a division loaded in this manner is ten, of which six at least must be 40 feet in length. The six 40-foot cars will carry six pontons, four chess, the tool and the forge. The remaining four cars, which may or may not be 40 feet, will carry the remaining two pontons and the two trestle wagons. A 40-foot car will carry three chess wagons, but if less than this length, only two.

In estimating the number of cars necessary to accommodate any given amount of equipment, as the total number of chess, tool, and forge wagons is always less than the total of ponton and trestle wagons, to use the minimum number of cars, the number of 40-foot cars required will be equal to the total number of chess, tool, and forge wagons. The number of additional cars of any length (not less than 34 feet) required will be the difference between the number of 40-foot cars required and the total number of ponton and trestle wagons.

It is not always possible to obtain 40-foot cars, especially when the movement is unexpected and railroad companies have had no advance notification. In this case, of course, more cars will be necessary. When all the cars are less than 40 feet a division will require ten for the ponton and trestle wagons and three for the remaining six wagons, a total of thirteen. In fact, the simple rule to follow is, the ponton and trestle wagons of the heavy train will each require a car, if these cars are 40 feet in length, any one of the other wagons of the train can be loaded on this car, otherwise not. A 40-foot car will carry two of the advance guard ponton wagons, otherwise the method of estimating the requisite number of cars for this train is the same.

As previously stated in this article, the Third Battalion of Engineers was directed to take with it to San Antonio two divisions of heavy and two divisions of light equipage. A large amount of this material, due to the lack of the necessary number of ponton sheds, was not on wheels. Balk, chess, and wagons were stored in the Engineer Depot, and in a manner more with a view to conserving space rather than the ease of removal and loading on cars. The wagons were knocked-down and piled on top of each other, and the chess and balk were in piles reaching from floor almost to ceiling. Then, too, track facilities were by no means sufficient to the demands of the situation. None of the sidings on the reservation were

convenient to the sheds, all the equipment had to be hauled a distance of about 600 yards to the cars. As there were not sufficient wagons set up, and as time was too limited to set up additional wagons, the method of loading described above could not be followed except with the last division. Even so, the loading of this material was accomplished in very short order; with proper facilities it could have been done in less than half the time it actually took. One valuable lesson was thus taught by the "maneuver" right at the start. A sufficient number of ponton sheds should be provided at the Depot to enable all ponton material to be stored when not in actual use set up on wheels. These sheds should have loading platforms and track sidings, so constructed that the wagons

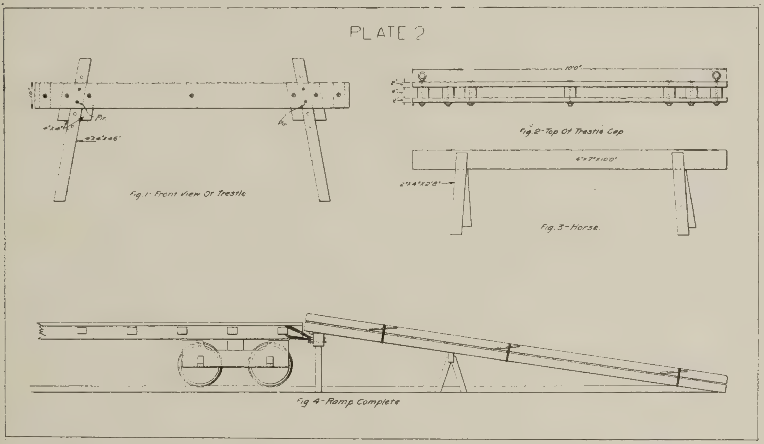


Plate II. Trestle for use in loading ponton equipage on cars.

could be run out of the sheds on to the platforms and from there directly on to the cars lying alongside. This arrangement would enable the loading to be carried on simultaneously at several different points, and the entire equipment could thus be placed on the cars in very short order. The need of expedition in this matter was forcibly impressed upon the author's mind by the character of the orders issued directing the recent mobilization at San Antonio. The entire equipment of a ponton battalion could be placed on cars in about two hours, if such an arrangement as suggested above were carried out.

The "maneuvers" at San Antonio have been very valuable to the officers of the ponton company. Although the amount of equipage brought was twice that assigned to a company by the Field Service

Regulations, the duty of handling the entire equipment was assigned to L Company, and this company was designated as the ponton company of the division. The other three companies constituted the pioneer battalion, which is the normal allowance of engineer troops, and although a division normally has no pontoniers, the proportion of pontoniers to pioneers by this arrangement was about correct.

Upon arrival at San Antonio, orders from division headquarters directed that the ponton train would not be removed from the cars. As the loading, due to causes before stated, had not been very systematic, a few cars more were requisitioned, and an entire rearrangement was effected. Everything was placed on wheels, and each division was isolated. The train, when completed, was then arranged so that the two heavy divisions were at the head, followed by the two light divisions.

All teams were turned over to the ponton company, and a requisition for one hundred additional mules was submitted. These animals, however, were never supplied.

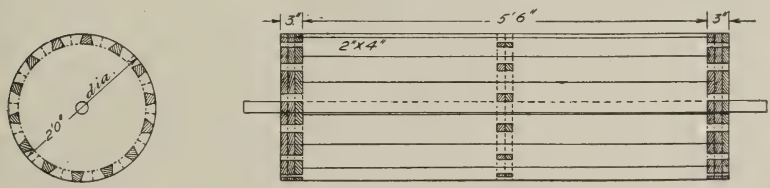
About the 1st of April all ponton equipage was removed from the cars and taken to camp. Systematic work was then begun to train teams and drivers. No bodies of water were available for the boats, so that drill for pontoniers was limited at first to lashings, setting up and taking down the canvas boats, and setting up trestles. The trains were taken out for marches of from 15 to 16 miles twice a week. On other days improvised drills were given for the benefit of the drivers; the train executed such movements as right and left front into line, right by wagon, by wagon by the right flank, and similar evolutions. The benefit from these simple drills was surprising. A set of raw teamsters was completely broken in and became proficient drivers in less than three weeks time. The writer believes that a standard drill for the ponton train could be readily devised, and that such a drill would be not only feasible but extremely useful. The drivers of a ponton train should be drivers in the same sense as in a battery of artillery.

After the boats had been out of the water for about three months and exposed to the sun during that time, they were found to be about as water-tight as sieves. It was realized that unless they were soon placed in the water, the division would have a ponton company only in name. On June 1, the company was detached and established its camp on the western outskirts of town near a

small body of water known as the West End Lake. Drills in reserve and advance guard bridge building, and rowing, both by ponton and by flotilla, were then taken up and have been regularly continued since that time.

In the flotilla drill the idea has been to substitute shorter and more intelligible commands for the unnecessarily long and obscure commands given in the Ponton Manual, and also to change in some instances the manner of execution of a few of the movements. For instance, instead of the command "By ponton from the right, front into column" the same movement may be effected by the short and simple command "Right by ponton," and, as an example of a modified movement, the right wheel was discarded, being replaced by the better movement "Right turn." As a matter of fact, the whole flotilla drill could be revised. As a drill in rowing, it is of great value and should by all means be retained; but it should be

PLATE 3



ROLLER

Plate III. Roller for use in loading pontons on wagons.

brought up to date and made more in keeping with modern drill regulations.

In bridge building a few changes in the methods described in the Manual have been instituted with a view to gaining time, and have been quite successful from that point of view. These changes, however, have been only in minor details, and such as would naturally suggest themselves. About the only thing of special interest that the company has accomplished in this line at San Antonio was an unusually long bridge built over the lake, shown in Fig. 2. The bridge was 850 feet in length. In its construction twelve trestles were used, eight of the Rees type and four Birago; twenty wooden boats, one steel boat, and one sectional steel. The bays with floating supports were built with extended intervals. The trestles were placed from a raft, as shown in Fig. 3. Of the three types used, the modified Rees type was most easily placed and most readily

adjusted. Of the three types used, this type is unquestionably the best; but even so, there is room for improvement. A perfectly satisfactory type of trestle is yet to be evolved. The writer has had no actual experience with the Ralston, but had the occasion to see an example of this type set up and tested. It is open to the objection, as are all the other types, that it is clumsy and difficult to place and adjust, especially in water. It is believed that a trestle all steel, or with steel legs and with a cap which, instead of having the legs passed through it, rests on movable supports attached to the legs, would be an improvement. Such a trestle

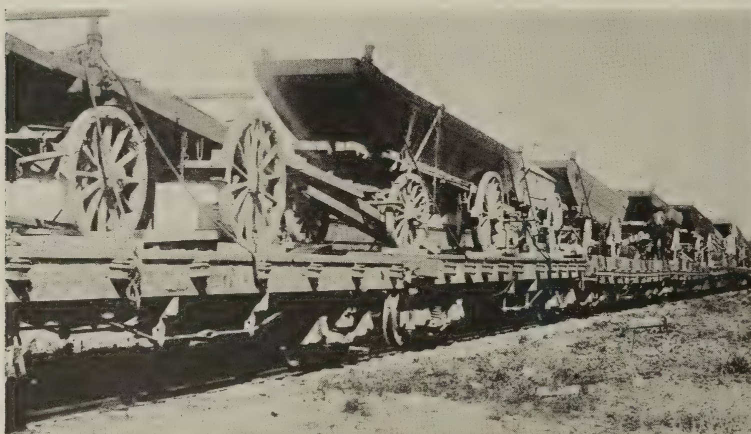


Fig. 1. Reserve equipage ready to ship, standing up.

could be constructed with less bulk, and possibly less weight than any of the types now in use.

In constructing that part of the bridge where pontons were used, on account of its great length, the method by parts was used. There was no current to contend with, so the parts were readily rowed into place with the assistance of one or two lines fastened to anchors previously cast. Fig. 4 shows a five-boat part being rowed into place.

Possibly the most valuable lesson to be learned from the construction of this bridge is to be gathered from observation and comparison of the steel and wooden boats. The author, up to this time, was inclined to favor the retention of the wooden boats. He is now decidedly in favor of the steel type, and for the following reason: Although the wooden boats had all been freshly caulked,

tarred, and painted in the previous March, and had been in water for six weeks, they all without a single exception, when heavy loads were brought upon them sufficient to throw the water line above seams that had heretofore been exposed, leaked very badly, some of them filling with water, and all requiring almost constant attention to keep them afloat. On the other hand, the steel boats that had not been in the water for over six months, leaked not a single drop, and needed no attention after being placed in the bridge.

Of the two types of steel boat, the one piece is superior to the sectional. The latter type is more difficult to handle in loading,

Fig. 2. Ponton Bridge, 850 feet long, constructed at West End Lake, San Antonio, Tex.

unloading and launching, and gives more trouble in transportation. The one-piece steel is about the same weight as the wooden, and handles with about the same ease. The particular boat now being experimented with has some serious drawbacks, but a board of officers, of which the writer is a member, is now preparing the plans for a one-piece steel boat, built on about the same general lines as the one now in use, but, it is hoped, with its faults corrected. If the ponton equipment is to be retained in our service, and it no doubt will be, a steel boat will eventually have to be adopted, and, in spite of all that may be said to the contrary, it will prove more satisfactory than our present out-of-date wooden type boat.

As a matter of fact, our ponton equipment is practically what it was during the Civil War. In forty years but few improvements have been made. As compared with the advancement made in all other lines of military endeavor, the ponton equipage is sadly out of date. True, there has been some haphazard experimentation from time to time, but little has been accomplished for the simple reason that no one recognized and established policy has existed to guide all efforts of improvement toward a certain goal. The ponton train has passed from hand to hand, one company has it for a while and passes it on to another, each one retaining it long enough for the men to acquire an indifferent degree of skill in rowing and bridge building.

Fig. 3. Placing trestles from a raft, West End Lake, San Antonio, Tex.

The Field Service Regulations have recently been revised. It is noted that a ponton battalion consists of three companies, with two divisions each. It is not known what considerations entered as factors in deciding upon this amount of equipage per company, but the writer is forced to the conclusion that such decision was more or less arbitrary. Two divisions as now organized, with the large number of animals pertaining thereto, is too much for an engineer company to handle. It would be much better to increase the present normal division by one or two ponton sections and assign one of these increased divisions to a company. Furthermore, it is reasonable to expect a ponton company to be able to do

its own pioneer work, such as repairing and strengthening bridges, repairing roads, and the like. In order to do this without delaying the march of the entire train, the mounted section and the pack train should by all means be retained. If this were done the amount of tools to be carried in the ponton tool wagon would be so reduced as to allow the adoption of one combined tool and forge wagon for a division instead of two separate wagons.

The author feels that his experience has been far too limited for him to presume to suggest here any particular lines to be followed out in reorganizing and improving the ponton equipage. It is believed that the first step would be the organization of a *ponton* battalion at some point, as Fort Leavenworth, with proper facili-

Fig. 4. Constructing a bridge "by parts" at San Antonio, Tex. Showing a five-boat part ready to move.

ties for making experiments and testing improvements, and with a suitable shop equipment for manufacturing every item of the equipage.

High Water Damages due to Levee Construction*

GEORGE F. ARCHER AND KATE C. ARCHER *v.* THE UNITED STATES.

OPINION.

BARNEY, J., delivered the opinion of the court:

This is a suit to recover damages for the ''taking'' of the lands of the claimant by the Government in the construction of the improvements on the Mississippi River. The claimant is the owner of a plantation containing about 6,000 acres in the State of Mississippi and situated within a bend on the Mississippi River and comprising nearly all of the land within said bend. Before the Government began its improvements upon this river the local authorities at different points along its course below Cairo, Ill., had built levees, the purpose of which was to prevent the river at its high stages from overflowing the lands in their rear. It does not appear that these locally constructed levees materially raised the flood of the river at any point, but that the surplus waters found their way through numerous openings into basins of large area, and thence into the Gulf. During periods of high water the lands of the claimant had always been subject to occasional overflow, but not to such an extent as to materially impair their value. The levees constructed by local authorities, and afterwards adopted and added to by the Government, as hereinafter stated, were located at places considerably back from the river so as to leave between them and the river, or as might be said between the levees on both sides of the river, lands which were not affected by them unless the flood tide of the river was permanently raised. The lands in question in this case belonged to that class.

Before the Government began its improvements the local authorities had built a levee, a fragment of which is shown upon the map following, and which it will be seen was situated somewhat back from the claimants' plantation and from the narrow neck of land joining his plantation to the mainland.

In 1883 the Government began the work of improving the navigation of the Mississippi River, and has continued it ever since. The plan adopted seems to have been to adopt the local levees as far as practicable from Cairo down, to build new levees in intervening spaces, and thus unite them into one complete system for raising the waters of the river by confining it within a narrower channel and preventing its partial escape into the basins before

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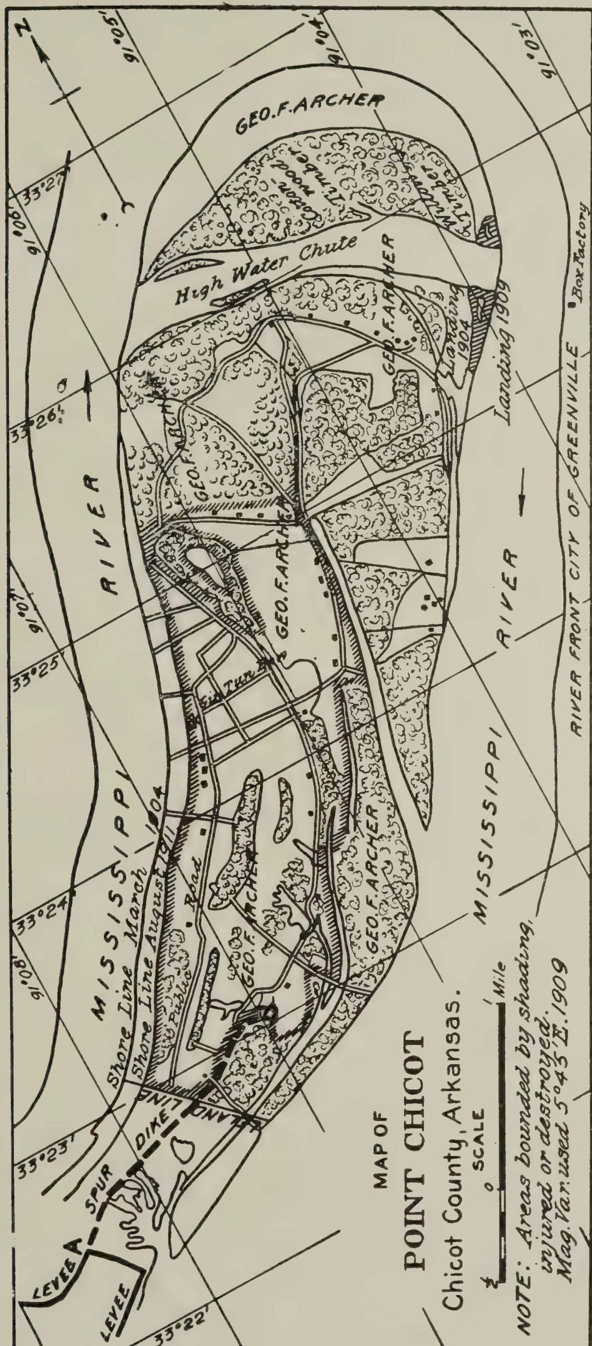
*Decisions in the United States Court of Claims.


MAP OF
POINT CHICOT
Chicot County, Arkansas.
SCALE
0
1 Mile
NOTE: Areas bounded by shading, injured or destroyed.
Mag. Var. used 5°45'E. 1909
N
GEO. F. ARCHER
High Water Chute
Landing 1904
Landing 1909
Box Factory
RIVER FRONT CITY OF GREENVILLE
MISSISSIPPI RIVER
MISSISSIPPI
RIVER
Shore Line March 1904
Shore Line August 1911
Public Road
SPUR
DIKE
LEVEE
LEVEE
33°22'
33°23'
33°24'
33°25
33°26½'
33°27'
91°08'
91°07'
91°06'
91°05'
91°04'
91°03'

mentioned. This object has been and is being further accomplished, and it appears that its waters have thus been raised approximately 6 feet in times of high water. This result has been to greatly improve and raise the value of much territory outward from this system of levees, but to cause more frequent overflow of the lands left between them and the river. It appears, however, that notwithstanding this apparent result of the river improvements the plantation of the claimant was sufficiently above the flood tide of the river to remain valuable agricultural lands and to be subject only to occasional overflow.

It appears that in 1903-1904, in order to prevent threatening danger to the levee near the neck of land connecting the plantation of the claimant and the mainland, as well as to preserve the integrity of the neck of land itself and prevent the river from cutting through it, the Government constructed a ''dike'' running diagonally out from the main line of the levee and extending 662 feet upon the Chicot plantation. In the period of high water of 1904 this dike was partially destroyed and the claimants' plantation considerably injured, whereupon in 1907 the Government extended said dike farther to the northeastward, and so far that it went 2,700 feet still farther into and upon the claimants' plantation. The effect of said dike at times of high water is to deflect the flood waters of the river to the eastward over and across a large portion of claimants' plantation in such volume and with such force as to scour out and destroy much of its top soil, wash away the tenant houses which were situated thereon, and leave a deposit of sand and gravel on its surface, which has already totally destroyed its value for agricultural purposes (and it has no other); and it is unnecessary to state that this deposit will increase from year to year.

This description of the *locus in quo* will be better understood by an examination of the map herein. The land to the northeast of the line marked ''Leland line'' is the farm of the claimant, the line colored yellow* is the levee, and the pink line A-B is the dike mentioned. The land submerged with sand and gravel is situated within the fretted dark line northeastward of the dike.

From this statement it will be seen that there is no question but that this land so destroyed has been ''taken'' within the meaning of the fifth amendment to the Constitution, and the only question before us is whether the Government is liable for this taking.

It is contended by the Government that it is not liable for this taking of the claimant's lands for the reason that the levees in question were partly constructed by the local authorities, so that the taking was not alone by the United States, but jointly with others. It is argued that such a joint taking is not a taking by the United States within the meaning of the amendment referred to. While the findings show that Point Chicot plantation was

*Levee and spur dike not colored, but marked by name in cut.

more frequently and deeper overflowed in consequence of the extension of the levee system by the United States, no part of it was destroyed nor materially damaged until the dike was extended and erected. Hence it can not be said that claimants' land was taken in consequence of the levee system, but directly in consequence of the erection of the dike, and that was the work of the United States alone. In this connection it should also be remarked that the levee system as constructed and kept up by the local authorities had not materially damaged these lands, and it was not until this system had reached "the state of completion which now exists" and said dike had been erected that they were inundated with sand and gravel (Findings II and VI). Hence, both by the construction of the dike and the completion and joining as a whole of this levee system the United States alone has taken the claimants' lands. This joining of the locally constructed levees and erection of the dike was in furtherance of the Government project alone—the confinement of the river within its banks and thereby to improve navigation. There may well be other cases where the overflow and damage are directly caused by the joint action of the Government and local authorities, and in such cases the question raised by the Government will arise and must be decided, but for the reasons given is not believed to be in this case.

It is further contended by the Government that this case comes within the decision of the Supreme Court in the *Bedford case* (192 U. S., 217), and that under that decision the petition should be dismissed. In that case a revetment was constructed by the Government authorities along the banks of the Mississippi River at Delta Point, La., for the purpose of preventing the further erosion of that point. As a result of this revetment the integrity of the bank at that point was maintained, and the channel and current of the river were gradually directed toward the lands of the claimants situated about 6 miles below, and after the expiration of several years eroded and overflowed about 2,300 acres of their lands. In the opinion in that case it is said:

" * * * The object of the works was to preserve the conditions made by natural causes. By constructing works to secure that object appellants contend there was given to them a right to compensation. The contention asserts a right in a riparian proprietor to the unrestrained operation of natural causes, and that works of the Government which resist or disturb those causes, if injury result to riparian owners, have the effect of taking private property for public uses within the meaning of the fifth amendment of the Constitution of the United States. The consequences of the contention immediately challenge its soundness. What is its limit? Is only the Government so restrained? Why not as well riparian proprietors; are they also forbidden to resist natural causes, whatever devastation by floods or erosion threaten their property? Why, for instance, would not under the principle asserted, the appellants have had a cause of action against the owner of the

land at the cut-off if he had constructed the revetment? And if the Government is responsible to one landowner below the works, why not to all landowners? The principle contended for seems necessarily wrong. Asserting the rights of riparian property it might make that property valueless. Conceding the power of the Government over navigable rivers, it would make that power impossible of exercise, or would prevent its exercise by the dread of an immeasurable responsibility * * *.''

In accord with the rule of law thus stated it would have been within the rights of the claimants to have constructed this dike if necessary to protect their lands from the ravages of the river, or for the Government to have constructed like works along the banks of the river for the improvement of navigation. But it seems to us an entirely different thing and not within the principle laid down in that case for the Government to invade the lands of the claimants and construct a dike thereon more than a quarter of a mile from the bank of the river, extending upon the same over 3,000 feet and covering more than 31 acres of land. It appears that this was done for the purpose of protecting the levee at the inner end of the dike, as well as to prevent the river from cutting through the narrow neck of land at that point, and if it had resulted in causing the overflow and taking of other lands than the claimants' situated farther down the river, the parties thus injured would have been without redress under the decision in the *Bedford case*. But in the case at bar it is admitted that the Government went upon the lands of the claimants and took possession of more than 31 acres, and constructed thereon a work which directly caused the taking of 3,664.6 acres more. We think it is not and can not be denied that the Government is clearly liable for the taking of this 31 acres; and if so, why is it not just as liable for the taking of the balance which was the result of the first taking? In fact, it may be said all of the lands mentioned were taken at the same time and by the same act.

We believe that the rule laid down in the recent case of *United States* v. *Grizzard*, 219 U. S., 180, is particularly applicable to this case. In that case there was an undisputed actual taking of a part of the farm of the defendant in error by permanently flooding the same, and, as an incident of such flooding, a public road running across the flooded part was also flooded. The court below had divided the damage by reason of the flooding of a part of the farm and the destruction of the easement, allowing $750 for each. The Supreme Court affirmed this judgment, and in so doing established the rule that ''Whenever there has been an actual physical taking of a part of a distinct tract of land the compensation to be awarded includes not only the market value of that part of the tract appropriated, but the damage to the remainder resulting from that taking, embracing, of course, injury due to the use to which the part appropriated is to be devoted.'' (Id., 183.)

As before stated, in the case at bar it is undisputed that the Gov-

ernment has taken actual physical possession of 31 acres of the plaintiff's farm, and the findings show that such taking has entirely destroyed the value of a large part of the remainder; and whether or not it is conceded that this absolute destruction of value by itself alone is a taking within the meaning of the fifth amendment of the Constitution, it will not be denied that this destruction of value resulted from the actual physical taking of the 31 acres. The same principle is recognized in *Sharp* v. *United States,* 191 U. S., 341, and *Welch* v. *United States,* 217 U. S., 333.

The case of *Barden* v. *Portage* (79 Wis., 126; 48 N. W. R., 210) is identical in principle with this case. In that case a levee was constructed on the plaintiff's land upon the Wisconsin River, and terminated at a point thereon, the result of which was to gather the waters of the river and deflect them at this point of termination over and upon the plaintiff's land and thereby cover it with sand and gravel. It was unanimously held in that case that the defendant was liable for damages thus ensuing.

To maintain the doctrine that lands can be invaded and a tract admittedly taken for the erection of a work to change the flow of waters, without subjecting the party so doing to the payment for all the lands thus taken, including such as are incidentally destroyed, would appear to us to be shocking to every principle of law and justice.

Judgment will be entered for the plaintiff in the sum of $47,000, but before the judgment is certified the rule stated in *Heyward* v. *United States,* 46 C. Cls., —, as to the survey and conveyance of the lands taken, and possible correction of the judgment, will be followed.

MATTIE W. JACKSON, Widow, *ET AL, v.* UNITED STATES.

OPINION.

Booth, J., delivered the opinion of the court:

This is one of a class of cases involving an alleged taking of private land in the course of the improvement of the Mississippi River by the United States in aid of navigation. The original petition was filed in 1894 in which the ad damnum was stated at $107,257.50. Defendants' demurrer to this petition was overruled in 1896 (31 C. Cls., 319), since which time three supplemental petitions have been filed increasing the aggregate damages claimed to $569,702.50. The lands of claimants lie on the east bank of the Mississippi River, about 40 miles below Natchez, in Adams County, Miss., embracing a total of 4,265.05 acres, segregated by description into four plantations, as follows: Cerro Gordo, Black Hills, Alloway, and Wakefield. The petition alleges that prior to the year 1890 said plantations were comparatively high and so situated as to be exempt from the overflow waters of the Mississippi River except at long intervals, and that the occurrence of such overflows did not materially affect their productive ca-

pacity or their value; that about 1883 the officers and agents of the United States, in pursuance of an act of Congress creating the Mississippi River Commission, and by subsequent acts passed to aid in the improvement of the navigation of said stream, have projected and constructed, and are now constructing a system of public works, consisting of levees and embankments, for the purpose of confining the flood waters of said river between the lines of said levees and embankments, thereby securing an increased elevation and force to the current of said river in order to scour and deepen the channel; that in the prosecution of said work by the officers of the commission, they have adopted and made use of the various State systems of public levees and private levees constructed for the reclamation of overflowed lands lying alongside the river wherever the same are available; that on the east side of said river from Vicksburg to Baton Rouge no levees have been constructed, the officers of the commission availing themselves of the highlands skirting the same and have adopted the lands between the levees on the west and the foothills on the east as the channel of the river, and the lands here claimed for lie therein. The petition concludes with a general averment that as a result of the adoption of the Eads plan, involving the reclamation of the flood waters of the river by the erection of levees and embankments and detouring samê into its channel, it has thereby increased its flood heights to such an extent as to annually inundate the premises of the claimants, destroying their value as agricultural lands, and leaving thereon a deposit of silt and sand of such proportions as to force their abandonment.

Claimants' contention rests entirely upon the assertion of a right under the Fifth Amendment to the Constitution of the United States, to compensation for private property appropriated by the United States for governmental purposes. The defendants interpose two defenses. First, that the damages accruing were consequential in character, and second, that the public works complained of were constructed by the cooperation of the United States and the various local authorities, with no means at hand to ascertain the extent of their respective liabilities.

The distinction between consequential damages occasioned riparian owners by the construction of governmental public works in navigable streams, and a taking of private property in furtherance of the same, is most generally a difficult and nice question of law. The rule is well settled that where officers of the United States appropriate to a public purpose the private property of another, admitting it to be such, an implied obligation arises to pay for the same. (*South Carolina v. Georgia,* 93 U. S., 4; *United States* v. *Great Falls Manufacturing Co.,* 112 U. S., 645; *United States* v. *Lynah,* 188 U. S., 445.)

In *Pumpelly* v. *Green Bay Company* (13 Wall., 166) the Supreme Court overruled a contention that a taking of private property within the meaning of the Fifth Amendment to the Constitu-

tion was limited to the identical lands physically taken, and extended the liability in such cases to other lands actually invaded by "superinduced additions of water, earth, sand, or other material * * * so as to effectually destroy or impair its usefulness." In the Pumpelly case the lands involved were totally submerged by overflow waters and had been so since the completion of the public works and for at least six years subsequent thereto; they were adjacent to the impediment placed by the defendants across the stream and were so situated that the result of the improvement was to retard the natural flow of the water and accumulate such a volume of same at the situs of the works as to back up the overflow upon and over the plaintiff's lands. It in effect amounted to a physical invasion and a practical ouster of the plaintiff's possession. To the same effect are *United States* v. *Great Falls Mfg. Co.* (112 U. S., 645); *United States v. Lynah* (188 U. S., 445); *United States* v. *Williams* (188 Ib., 485).

In the Lynah case, *supra,* a case especially relied upon by the claimants, the findings show, and from the opinion it is clearly deducible, that it is not a departure from the previous rulings of the court upon this subject. Mr. Justice Brewer, in speaking for the court, says: "It is clear from these findings that what was a valuable rice plantation has been permanently flooded, wholly destroyed in value, and turned into an irreclaimable bog; and this as the necessary result of the work which the Government has undertaken." While there was dissension as to the full import of the findings, there was no dispute as to the rule of law. Again it is observable that in the Lynah case the improvements complained of were placed in the bed of the river having the same disastrous effects as in the Pumpelly case.

In *United States* v. *Welch* (217 U. S., 333) and *United States* v. *Grizzard* (219 U. S., 180) the Supreme Court extended the quantum of compensation recoverable for an actual physical taking of private property under the Fifth Amendment so as to embrace not only the market value of the lands actually taken, but to the remainder affected by such invasion, including the right of access to a public road destroyed by permanent flooding.

While what constitutes an actual taking is difficult of ascertainment, it is clear from the opinions that to constitute an actual taking there must be an actual invasion of the lands amounting to a practical ouster of claimant's possession, an actual overflow of such a permanent character as to imply an intent to take, and a correlative obligation to pay for the lands so taken. *Peabody* v. *N. S.* (43 C. Cls., 5). The Supreme Court has said that "the acts done in the proper exercise of governmental powers, and not directly encroaching upon private property, though their consequences may impair its use, are universally held not to be a taking within the Constitutional provision." In *Transportation Company* v. *Chicago* (99 U. S., 635)—from which the above quotation is taken—the court held the municipality exempt from liability for

damages unavoidably caused an adjoining property owner by obstructing a street and a portion of the river in the course of constructing a tunnel under the Chicago River.

Numerous decisions covering the entire scope of consequential injuries as distinguished from a taking under the Fifth Amendment of the Constitution, are discussed and cited in the case of *Heyward* v. *United States* (46 C. Cls., ——); it is unnecessary to again discuss them here.

In *Bedford* v. *United States* (192 U. S., 225)—a case of extreme significance to the issue here raised—the Supreme Court in distinguishing the difference between consequential damages and a taking of private lands for public purposes, declined to attach responsibility to the Government for constructing certain improvements in the Mississippi River in such a way as to result in a complete and permanent submerging of certain portions of the claimant's lands. The findings of the court in the Bedford case disclose the following situation: Prior to 1876 the channel of the Mississippi River was around a narrow neck of land known as De Soto Point; in the spring of that year De Soto Point, yielding to constant erosion and the force of the current of the river, became so narrow that the river broke through, thereby detouring the main channel from in front of the city of Vicksburg to a distance some miles away in a southerly direction. The effect of this complete change in the channel of the river was to force the water with great velocity against the Mississippi bank at what is known as the cut-off of 1876. The United States in 1878 and subsequent years, in pursuance of acts of Congress, erected along the new banks of the river near the cut-off some 10,700 feet of revetments, the purpose being to prevent further erosion of the banks of the new-made channel, which, if continued, would necessarily carry the main channel of the river farther away from the city of Vicksburg. In the consummation of this purpose and because of the revetment work the waters of the river were deflected toward the land of the claimant, situated some 6 miles below the same, and subsequently permanently submerged them. In answering the plaintiff's contention, the opinion uses the following language:

"The contention asserts a right in a riparian proprietor to the unrestrained operation of natural causes, and that works of the Government which resist or disturb those causes, if injury result to riparian owners, have the effect of taking private property for public uses within the meaning of the Fifth Amendment of the Constitution of the United States. The consequences of the contention immediately challenge its soundness. What is its limit? Is only the Government so restrained? Why not as well riparian proprietors; are they also forbidden to resist natural causes, whatever devastation by floods or erosion threaten their property? Why, for instance, would not, under the principle asserted, the appellants have had a cause of action against the owner of the land at the cut-off if he had constructed the revetment? And if the

Government is responsible to one landowner below the works, why not to all landowners? The principle contended for seems necessarily wrong. Asserting the rights of riparian property it might make that property valueless. Conceding the power of the Government over navigable rivers, it would make that power impossible of exercise, or would prevent its exercise by the dread of an immeasurable responsibility.''

We have given at length and in great detail the substantially agreed upon findings respecting the hydraulics of the Mississippi River. The evidence upon which they are predicated consists of the numerous reports of the Mississippi River Commission and various other reports of the authorized officers of the Government in the course of said work. From said findings it is apparent that said stream from Cairo, Ill., to the Gulf of Mexico is one of great sinuosity; its innumerable bends with scarcely a single line of direct current have made it susceptible to great overflows in times of anything like abnormal conditions. In fact, overflows are so frequent, and the use of riparian lands for agricultural purposes so precarious, that it is indispensable to protect them by lines of levees and embankments. The delta of the river extending from Girardeau, Mo., to the Gulf of Mexico has been divided into six large basins (four on the west bank and two on the east); through the medium of these large and extensive formations the flood waters of the stream have from time immemorial been discharged, passing consecutively from one to the other until they reach the Gulf. Within the limits of the respective basins are millions of acres of alluvial lands which have been at least partially reclaimed by the construction of private levees or their inclusion in local levee districts formed under local laws. These vast drainage basins in their natural state have in themselves been of inestimable value to the riparian owners of lands not situated therein, for the flood waters of the rivers escaping through them rapidly absorbed the surplus waters suddenly projected upon the higher lands and saved them from extreme injury.

The lands here involved are situated at the southeastern limits of the Lower Tensas Basin opposite what is known as the Bougere Crevasse. They are not protected or reclaimed by levees and lie in that extensive zone some 253 miles in length extending from near Vicksburg, Miss., to Baton Rouge, La., on the east side of the river where the Government has not seen fit to construct any levees or embankments or any other improvements to aid in the navigation of the river, the nearest Government levee to claimants' land being on the opposite side of the river in the State of Louisiana and the nearest Government levees on the east bank, one being 157 miles north and one 96 miles south in the State of Mississippi. They are alluvial lands situated within the Delta of the river, and have been and of necessity must have been subject more or less to the overflow waters of the river. It is conceded that there is nothing peculiar in their location and that they have been subject to over-

flow in times of high water. They lie, it is true, between the banks of the river on the west and the so-called highlands or foothills of the river on the east. The Bougere Crevasse occurred during the flood of 1859, and until it was subsequently closed served in part at least as a channel through which the flood waters upon claimants' lands were speedily reduced.

The United States in the creation of the Mississippi River Commission and the numerous appropriations granted to forward the work, contemplated a most comprehensive scheme involving the reclamation of the flood waters of the river, and by a system of levees and embankments erected upon the banks of either side of the same to prevent its overflow, confine this enormous volume of water in the main channel of the river, thereby securing an increased velocity to the current, which would eventually deepen the channel. The mere assertion is sufficient demonstration that as a result of this project the flood heights of the river would be materially increased, for it is quite apparent that the enormous volume of water previously escaping through these immense basins having been deflected into the main channel of the river would result in causing lands unprotected by levees or embankments to be subject to more frequent and indeed more serious inundations. In the prosecution of this general plan the United States have made use of and are now using the levees available for the purpose which were constructed by private owners of land or by State and local drainage districts. They have also connected this necessarily disjointed system of levee improvement by constructing new levees and embankments where none theretofore existed, until at present they have substantially a continuous line of levees on the east bank of the river from some distance south of Cairo, Ill., to the Gulf of Mexico, and at such points on the west side as in the judgment of the engineer officers of the Army serve the purposes of the improvements.

The findings show, and it is conceded, that as a result of the system employed by the United States, in connection with the State and local authorities, the lands of the claimants have been and are now more frequently overflowed than before the construction of the levees. It is indisputable that a large portion of claimants' plantations have been practically destroyed for agricultural purposes by additional superinduced deposits of silt and sand of sufficient depth to render some portions of them valueless. It is not questioned that the claimants involved have suffered great loss in their inability to annually harvest crops or cultivate to maturity the products usually raised upon said lands.

One contention of the claimants extremely vital to the case, set forth in the petition and emphasized in the briefs, fails for want of proof. It is this, that the Mississippi River Commission has adopted as the main channel of the river from Vicksburg to Baton Rouge the lands between the levees on the west and the highlands on the east, and for this reason have not constructed any levees or em-

bankments on the east side of the river. To sustain this contention the court must indulge an inference from the general plan of the public works. There is an utter absence of any such express intent found in the numerous reports of the commission. The officers of the commission have upon numerous occasions in their reports urged upon Congress some equitable legislative relief for the numerous sufferers in this particular locality, and have described in detail their unfortunate situation and predicament, but we have been unable to find (and certainly cannot conjecture) that it was part of general plan of improvement to appropriate as the channel of the river this most extensive area of private lands extending along the river bank to a total length of some four or five hundred miles, and increasing the width of the channel in some instances more than a mile. The damages would indeed be immeasurable and the court could not sustain the judgment asked for in the absence of strong and convincing proofs. The testimony to sustain such contention, if it could be sustained, is easily accessible from living witnesses, and so clearly subject to positive proof that inferences and implications from other testimony in the record are unwarrantable.

The Bedford case establishes that the United States in the exercise of its plenary power and authority over the navigable streams of the country in aid of commerce and navigation can by public works resting only against the banks of the channel prevent the same from erosion and preserve its natural identity; that consequences however injurious resulting from such procedure are but natural results, consequential in character, and *damnum absque injuria*. The improvement of the Mississippi River through the instrumentality of a congressional commission manifestly purposed not only the reclamation of the extensive flood waters of the stream, but the erection of such permanent structures along its banks as would prevent the same from erosion and successfully resist the increased velocity of the current and the increased flood heights of the river. The Government was not concerned in the reclamation of riparian lands and was without authority to expend money for the purpose. (Act Mar. 3, 1881; 21 Stat., 468-474.) It was alone concerned in an endeavor to establish settled conditions, throw the escaping flood waters back into their natural channel, and keep them there. It undertook to preserve the channel of the river, the channel the river itself had made in its meanderings from its source to its mouth.

The claimants' lands, unfortunately situated as were the lands of Bedford, suffered from this improvement in that they were more frequently overflowed than theretofore and the resultant deposits were more extensive.

The findings show, and it is conceded, that said lands are not and never have been permanently submerged; that in the years 1894-1896, 1900-1902, 1905, and 1910 they were not overflowed at all; that despite partial overflows from 1896 to 1908 the claimants have harvested and sold \$328,003.98 worth of cotton therefrom;

that as late as the season of 1909 claimant E. H. Jackson had 500 acres in cultivation, and claimant Mattie W. Jackson in 1910 was enabled to realize profit from her plantations which were not overflowed. Aside from the question of permanent submerging, even if same prevailed, the claimants under the authorities cited could not recover. The United States was clearly within the scope of its authority in preserving the banks of the river, and if thereby the perpetual continuance of the great basins of drainage made by the overflow waters of the river which had served as natural outlets for the same were destroyed, it was but the incidental result of the prosecution of the work and the United States is not to be held liable in damages for pursuing its general plan of improvements alongside the established channel of the river whereby it prevents the water which should be in the channel from escaping elsewhere.

This case is not like the case of *Barden* v. *City of Portage* (79 Wis., 126); no artificial structures were placed on or near the claimants' lands; no waters were deflected toward the same; the public works complained of simply destroyed their existing means of drainage made by the uncertain flow and course of an exceedingly crooked and unreliable water course. Prior to 1859 claimants had no outlet through the Bougere Crevasse. There was no absolute certainty that it would continue to be a means of drainage for the lands, for an unusual flood height, a sudden change in the elevations of the basins, or the making of a new channel by the river itself might have destroyed its usefulness and thereby subjected claimants to injuries as extensive as here claimed for. The United States closed the crevasse by the construction of levees on the banks of the river and the flood waters theretofore escaping through this channel were retarded and remained longer on the claimants' lands, just as in the Bedford case the United States held intact the new-made channel of the river and thereby submerged 2,300 acres of the plaintiff's lands which would have remained high and dry if the water had continued in its old channel. The fact that claimants' lands were not so frequently subject to overflow under the natural conditions that existed prior to the construction of the levees does not obligate the Government, in the lawful prosecution of public works in aid of navigation and commerce, to avoid a disturbance of those natural conditions or otherwise incur extensive liabilities.

The facts in the case of *Archer* v. *United States,* No. 30471, decided December 4, 1911, are so entirely different from the facts in this case, the decision of the court in that case can not apply here. In the Archer case the findings show that the officers of the United States, to protect the channel of the Mississippi River, actually invaded and took possession of more than 31 acres of the lands of Archer and constructed thereon a spur dike, made out of his own soil, some 662 feet in length. The result was to deflect the current of the river over and across the lands of the claimant, in consequence of which they were rendered valueless. The Archer

case is similar in most respects to *Pumpelly* v. *Green Bay Co., supra,* and *Lynah* v. *United States, supra.*

The great basins of the Mississippi reclaimed the lands of riparian owners on the opposite sides of the river from where they were formed and forced those within their limits to erect levees and embankments or abandon their farms for cultivation. The public works of the United States in the aid of navigation incidentally closed these immense outlets, not in this case by a physical invasion of claimants' property, not by appropriating any portion of their soil for levees, nor by proceedings in eminent domain, but by a system of levees built and adopted where previously built on the banks of the river to prevent the water from getting out of the channel and becoming so low as to impede and retard navigation. The bed of the stream was not disturbed; no dams or cross-tide dams, jetties, or other improvements retarded the flow of the water and backed it up and upon claimants' lands. The United States simply took the banks of the river as they found them and sought to preserve them *in statu quo.* The condition now is what it would have been had the overflows been restrained long years ago. The character of the work done was not essentially different from dredging; without doubt the governmental authorities had full power and authority to deepen the channel by dredging, and if they adopted a different means better suited and perhaps more inexpensive, which in effect accomplished the same purpose, the results are the same. Surely it could not be said from the adjudicated cases that the United States is disabled from increasing and preserving from erosion the banks of a navigable stream and thus forestalling by an important improvement the continuance of a condition which if allowed to continue would eventually destroy the usefulness of the river as a commercial highway without incurring, as was said by the court in the Bedford case, "immeasurable responsibility." Claimants' lands from their natural state were burdened with the servitude of a dominant right in the Government of the United States to improve the river in aid of navigation and commerce.

The Mississippi River Commission, in its annual report for 1894 at page 2713, reviews at length the subject of injuries to private lands situated in the alluvial basins of the river. The whole tenor of their observations indicate an apparent indecisiveness as to the extent of responsibility attaching to the United States and the State and local authorities. In speaking of the erection of private levees by the owners of riparian lands in this particular locality, whereby the same could be reclaimed and protected, the commission uses this language:

"It must be recognized that the result will be to inflict some and perhaps great hardships upon the owners of lands in the unprotected areas described. Just how great the increase of burden cast upon those lands from this cause will be can not now be foreseen. They have always been liable to overflow by the highest

floods, and they have always escaped overflow in some years. It is probable that this will continue to be true in the future as in the past. There may be, however, some floods which, unconfined, would not overflow them, but which, confined, will overflow them, and the injury in such case would doubtless be of that immediate and proximate character which constitutes recognized ground of legal redress.

"But the subject is one with which the commission does not feel authorized to deal. In making recommendations for the expenditure of money in the construction of levees it has felt bound to make such application of it as would probably secure the largest aggregate of beneficial results. Some of the minor areas mentioned are large and valuable enough to warrant the expenditure of money necessary to protect them by levees, while others are not. As to the former, the work is at present simply deferred to await the completion of other work which is considered more important. As to the latter, the construction of levees by the United States would seem to be an expenditure of money merely or mainly for the purpose of repairing a private wrong. This the commission regards as beyond its jurisdiction."

From the report it would seem that it is not impossible for claimants to protect their lands from overflow by private levees and embankments, and Finding XXXIX shows that it had been done. If so, the duty is cast upon them and the damages claimed thereby materially minimized, if not fully prevented. (*Manigault* v. *Springs*, 199 U. S., 473-483.)

It is difficult to see from the record in this case wherein the improvements constructed by the United States on the banks of the Mississippi have resulted in such an invasion of claimants' lands as to amount to a practical ouster of possession. True, they are not in all respects as they were previous to the improvements, and doubtless their cultivation and value have been impaired. No doubt when they were purchased by the present owners a change in the situation as it then existed was not contemplated, but the ownership of riparian lands on navigable waters is always subject to the consequences of governmental improvement of the stream in aid of navigation. (*Gibson* v. *United States,* 166 U. S., 269.)

The rulings of the Supreme Court in the Bedford case alone preclude a judgment for the claimants, and the petition is dismissed. It is so ordered.

James Chatham Duane

James Chatham Duane (see frontispiece) was born in Schenectady, N. Y., on June 30, 1824. He was a son of Judge James Duane, a member of the Continental Congress and first mayor of New York City after the close of the Revolution, and a member of the Convention which adopted the Constitution of the United States.

He graduated from Union College in 1844 with the degree of A. B., and in July of the same year entered the United States Military Academy, from which he was graduated and commissioned a Brevet Second Lieutenant in the Corps of Engineers, July 1, 1848. He served at the Military Academy from 1848 to 1854 in the Department of Practical Military Engineering and was promoted Second Lieutenant, Corps of Engineers, March 16, 1853. From 1854 to 1856 he was assistant engineer on the construction of Fort Carroll, Maryland, and was promoted First Lieutenant, Corps of Engineers, July 1, 1855; from August, 1856, to April, 1858, he served as Light-house Engineer of the New York District, and from the latter date was in command of the Engineer Company in the Utah Expedition until October 13, 1858. He was then sent to the Military Academy as Instructor in Practical Military Engineering where he served until January 18, 1861.

At the outbreak of the Civil War he was placed in command of an Engineer company at Washington, D. C., and was engaged in the defense of Fort Pickens, Florida, and in the defenses of Washington. He declined a commission as Captain in the 12th Infantry, May 14, 1861, and was promoted Captain, Corps of Engineers, August 6 of the same year. During the period in which he was stationed in Washington, he was engaged in organizing the Engineer Battalion and Engineer equipage for the Army of the Potomac and the Department of the South. He constructed the bridge across the Potomac at Harpers Ferry in February-March, 1862. He participated in the Virginia Peninsular Campaign with the Army of the Potomac, commanding the Engineer Battalion in the Siege of Yorktown, and in the construction of fieldworks and bridges during the Seven Days' Battle. During the Maryland

Campaign he was Chief Engineer of the Army of the Potomac, and took part in the Battles of South Mountain, Antietam, and the various skirmishes in the gaps of the Blue Ridge. Until June 13, 1863, he was Chief Engineer of the Department of the South and took part in the attack on Fort McAllister, Georgia, and in the operations against Charleston, S. C.

He was promoted Major, Corps of Engineers, March 3, 1863, and served as Chief Engineer of the Army of the Potomac from July 15, 1863, until the end of the war, being engaged in the actions at Manassas Gap, Rappahannock Station, Robertson's Tavern, Battles of the Wilderness, Spottsylvania, Cold Harbor, Siege of Petersburg, Hatcher's Run, and the final pursuit of the Confederate Army.

He was brevetted Lieutenant-Colonel and Colonel, July 6, 1864, "For meritorious and faithful services in the Campaign From the Rapidan to James River, and Particularly for Distinguished Professional Services Before Petersburg, Virginia;" and was again brevetted Brigadier-General, U. S. Army, March 13, 1865, "For Gallant and Meritorious Services During the Siege of Petersburg and the Campaign Terminating with the Surrender of the Insurgent Army under General Robert E. Lee."

At the close of the War he was placed in command of the Post of Willetts Point, N. Y., and was Superintending Engineer of the Defenses at the Eastern Entrance to New York Harbor until 1868, having been promoted to Lieutenant-Colonel, Corps of Engineers, March 7, 1867. He was in charge of the fortification and light-house engineer duties at Portland, Me., from 1868 to 1879, and was Light-house Engineer at Tompkinsville, N. Y., from 1879 to 1886, having been promoted Colonel, Corps of Engineers, January 10, 1883. In 1884 he became senior member of the Permanent Board of Engineers for Fortifications and for Rivers and Harbors, and served on that duty until he became Chief of Engineers in 1886. He served in this capacity until retired for age in 1888. Shortly after his retirement he was appointed, by the Mayor of New York City, member and president of the Croton Aqueduct Commission, and occupied this position until his death in 1897.

General Duane was the author of a Manual for Engineer Troops and, in conjunction with General Abbot and Colonel Merrill, he prepared the Organization of the Bridge Equipage of the United States Army, published in 1870.

He died in New York City, December 8, 1897, at the age of 73 years.

Royal Engineers in Cooperation with Other Arms*

BY

Brig. Gen. F. C. HEATH, C. B.
Inspector of Royal Engineers

I have been asked by General Lawson to talk to you to-day on the employment of Royal Engineers with other arms and kindred subjects. This I am glad to do, because, whatever we may have been accused of in the past, I feel sure you will bear me witness that we are now as keen as any to come out of our shell of exclusiveness and work in with other arms.

The rôle of the Royal Engineers is to "help on the show," and this, of course, they can not do unless they know and thoroughly appreciate what other arms want, but besides this those other arms must realize the nature of the assistance that can be given, lest there be danger of their not asking for what they can get or of making impossible demands; so you see, we must know one another and more than this, the knowledge must be so intimate that help is given, or assistance asked, as a process of intuition. We can not afford to wait until the time of active service to learn these things, as then everything must go as if by clockwork.

My subject is a dry one I fear, but I trust it may help us to think out these matters.

I dare say most of you remember the days when a field company did not form part of the peace organization of a division, indeed, it is only some few years since all that was changed. In former days it usually happened that a field company was thrown at a G. O. C.'s head just as he was starting for a field day, with the obvious result that he and his staff uttered a bit of bad language and wondered how they were meant to use those d——d sappers.

But now that has all changed, and every division has, you know, as part of it a little battalion of Royal Engineers, consisting of two field companies and one divisional telegraph company, commanded by a lieutenant-colonel. The field companies never change stations; the lieutenant-colonel usually commands them for four years, and the other officers are with them for not less than three years. They are entirely under the General of Division and his staff for training, so you see it is now their fault if these engineers are not trained to suit their requirements.

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I am thankful to say that this new organization has worked a very great change, we are now being taught something, and the uses to which we can be put are being thought out and considered.

Now, if I can, I should like to enlarge on this matter and explain to those who have not thought out the subject some of the many ways in which we should be able to help for the common good.

To do this we will follow out an imaginary campaign. The war begins with a concentration at an oversea port or elsewhere, such as took place at Alexandria, for instance, after the bombardment of 1882. In this particular instance, the civil population had fled and there was no local skilled labor to be had. Now, on paper, a concentration at a well-found port seems simple, and, perhaps, looking at it from outside, it is so, but it is an extraordinary fact, human nature perhaps, that no one ever seems satisfied with what he has got. The landing stages are not convenient for landing some particular kind of stores and must be altered; special landing stages are wanted for troops, so that stores and troops should not get mixed up; the departmental staff find excellent sheds to accommodate their stores, but the doors and openings are all wrong, railway sidings have to be brought up to them; water must be laid on to the enclosure selected by the veterinary officer for his remounts; the medical officer and headquarter clerks vie with one another in their urgent demands for shelving, the one for his bottles and the other for his stationery. Believe me, the unfortunate R. E. gets inundated with requisitions of all sorts; some of the greatest importance, others only of importance because of the high quarters from which they originate; an active A. D. C. is sometimes a terror in these matters.

Now, in the early stages, just at the rush of things the only R. E. landed are probably the field companies and the only source from which tradesmen would be forthcoming to tackle these innumerable jobs. I want to tell you all this, because I have heard it stated that skilled tradesmen are not required in a field company, that a less expensive individual would suffice. Well, I may be prejudiced, but when I look up my journals and realize what calls were made on our tradesmen whenever a force was halted, either at the landing stage, on the line of march, or on the occupation of an enemy's town, such as Cairo, I am appalled with the thought of how blue the atmosphere would have been had tradesmen not existed. Even in the occupation of a friendly town it is wonderful how little accommodating the town seems to be, and how particular G. O. C.'s and C. O.'s become. But it is so natural that all these requisitions should be met, that no one waits to enquire what they entail to the R. E., and then they consist of such a multitude of small, though essential, services of which no record is kept, that no wonder when official histories are written no mention is made of them. I do not mean that we want this recognition, but what I wish to point out is the fact that when we are told that we do not

want tradesmen, we find it difficult to prove the necessity to those who have not had the experience of what is expected of us in these matters.

Well, then, from what I have told you, I hope you will admit we want tradesmen and that you will agree that a nominal 150 in a company is none too many.

Now, married men amongst you who are not in occupation of Government quarters and have to pay for their own repairs will no doubt remember their economical spirit getting a shock when it is a case of repair, a simple burst pipe for instance, the plumber with at least two assistants turning up for the job. Well, I am not prepared to say that both assistants are absolutely necessary, but certainly one is, and you bless your stars in thinking that after all he or they are only laborers and therefore cheap. So you will readily understand that it is not economical to use any of these 150 skilled tradesmen as assistants, for assistants unskilled labor is the cheapest, and the more there is of this, within reason, the more skilled work you will get out of your 150 sappers.

Perhaps some of you have guessed what I am leading up to; it is, of course, the necessity for working parties. So, when you are halted at the base, on the line of march, or elsewhere, where there is work to be done, let us have what labor you can spare ungrudgingly, because what work is done is done for you, not for ourselves. In foreign conscript armies, you know there are pioneer battalions, these I have always understood to be a self-contained units—that is, a units consisting of the necessary skilled tradesmen plus necessary unskilled labor. Until we have our own labor tacked on to our field companies like this, I feel we shall be obliged to borrow. In Egypt and South Africa we were allowed to hire natives, and in this way formed little pioneer battalions for ourselves, but that was exceptional.

Up to now I have been assuming that the army is more or less at rest, when the field companies would not, perhaps, be so closely associated with their divisions, but would, perhaps, be concentrated for, what I call, war works services, such services as I have mentioned above and those under the following six headings, which will probably appear in our new training manual, viz:

1. The improvement of the piers, wharfs, derricks, pier and wharf accommodation, exits from the docks, etc.
2. The provision of suitable entraining accommodation.
3. The improvement and making of roads in the docks and town, and the marking of the routes to camps and depots.
4. The provision of water supply and sanitary services for billets and camps.
5. The conversion of such buildings as may be required into hospitals, stores, sick lines for animals.
6. The erection and maintenance of suitable telegraphic or telephonic communication.

Plenty of work, you see, for the engineer companies and their tradesmen.

But the army is now to move; each division must draw in its engineers and move off complete, the engineer work at the base being left to the line of communication troops. The march begins, in the early stages out of reach of the enemy, when the principal duty for the field engineer will be that common to everyone else, viz: to do the marches with the minimum of fatigue to men and horses, and it will much assist in this if, at the end of the day's march, the force can go straight into a fully prepared billeting area or other accommodation.

In small ways there is a good deal to be done before a billeting area, camping, or bivouac ground can be said to be prepared. Water arrangements have to be looked into; the municipal authorities can often help a good deal in this matter if given time and treated with tact and consideration; then there are always gaps to be made through hedges and walls, gateways to be enlarged, swampy places to be improved, etc.; so you see if you want to find everything comfortable for you when you march in, I would advise you to keep in touch with the C. R. E. and get him to make an early reconnaissance with your staff officer, and, above all, get your field companies on the ground and at work as early as possible; if the enemy is not in close proximity, it might often help to send on a field company well in advance of the column, so as to be on the spot several hours before the troops arrive with the engineer officers, employed on reconnaissance, well in front of these again, so that they may meet the company with a detailed scheme and lose no time in getting the R. E. to work. I want here to impress upon you all the *supreme* importance of engineer reconnaissance; if you think a little, you will see how much depends on this. Before you can commence work, you must first know what you have got to do. You have to get the necessary tools and materials on the spot, think out the best way to do the work, the number of men required to do it, etc., and all this takes time, and unless the problem is cut and dried by the time the workmen arrive, there will be waste of time and much cussing and swearing, marching and countermarching before the particular tradesmen with the particular tools and materials they require meet one another at the desired spot. If you want us to do good work, you must help us to look ahead; the more we can do this, the more efficient you will find us. You must give us the very earliest intimation. This seems common sense, but yet, strange to say, there are occasions when this has not been given, and we have been expected to get work done as if we had a magician's wand at our disposal.

Engineer reconnaissance takes many forms. In my opinion, and I hope it will be clearly laid down in our new training manual, engineer officers should always accompany the general staff reconnaissance of a position or of a river or pass, or one made prepara-

tory to a march for the purpose of selecting billets, camps, etc. The duty of the engineer officer will be to study the best method in which engineers may be employed, so as to enable the commander of the engineers to advise on technical matters. It is important that reports, giving the details of engineering work required, should be sent from the cavalry to the army in rear, from the advance guard to the main body, and from the army to the lines of communication. It is only by the very early receipt of such information that the engineers with each portion of the force will be able to carry out the necessary work efficiently and promptly.

I remember not so very long ago at a staff ride, or divisional maneuver, it being a question of the passage of a river, staff officers were sent out to reconnoiter suitable places to cross. When these came in, the engineer officer was informed that he was to make bridges at the places specified. He had then to make his own technical reconnaissance, and, at least at one place, found the technical difficulties so great that the general staff officer had to go out again and find another place. I merely mention this to show how much time is really saved by the staff taking an engineer officer with them on this sort of reconnaissance.

Then there are reconnaissances in attack and defence. I will again quote them from what I hope may appear in our training manual shortly to be issued:

"Prior to, and during, an attack, the attention of the engineers should be specially directed towards such points as will facilitate the advance of the other arms. Reconnaissances will, therefore, be carried out with a view to ascertaining what obstacles will be met with, and how they can be crossed or destroyed, what improvements in communication and approaches are necessary, the work required to strengthen covering positions, and the best arrangement of telegraphic or other means of communication."

So you see there is a good deal in this matter of reconnaissance for the engineer officer, and I trust that those responsible for training will see to it that we do not neglect this important work.

On maneuvers, etc., it is one of my duties to watch the work of the engineers. When I meet a field company, my usual remark is "Well, what are you doing?" and I am sorry to say that the usual answer is "Oh, the usual thing, nothing; we have got no orders." I am afraid that I then get angry and want to know why there are five engineer officers on the roadway doing nothing. I want to know why the officers are not ahead looking to see whether they can not do some useful work. If there is nothing better for them to do, they could with advantage visit farms, sheds, etc., and see whether there is barbed wire and what tools can be got in case these are wanted later. And here I may remind you that an engineer company carries only some 108 pickaxes and each infantry battalion only some 155 pickaxes, rather more shovels in each case; no materials, such as barbed wire or timber, are carried. So you

see if entrenching is to be seriously taken in hand, it will be necessary to collect more picks and shovels from elsewhere, not to mention wire for entanglements and timber for overhead cover, and for these the engineer officer or others must search.

Well, we are now beginning to approach the enemy; you know it, because you find marching not so simple, the enemy is trying to delay you with obstructions, broken bridges, etc. If your cavalry is in front of you, and the engineer officers with the field troops accompanying it have done their duty, they will have sent back precise details of the nature of the obstructions, broken bridges, etc., which would enable the C. R. E. to make his plans and get to work as soon as *you* get him to the spot. His men have to march remember, and if they are far behind in the column you will have difficulty in getting them to the front and, when you have done so the men will have a forced march, so to speak, and you will not get such good work out of them than as if they had been with the advance guard. Incidentally, I might mention that some French authorities make a great point of relieving the sapper of as much weight as possible when on the march so that he may be fresh to undertake work when required. My own view is that during the march the greater part, if not all, of your engineers should be with the advanced guard, but, in every case, please take your C. R. E. into your conndence before your operation orders are got out and make him responsible for the distribution of his units. He should know best what men and tools are required for the job in hand. R. E. field companies are small units and take up but little road space, particularly if the pontons are relegated to the 2d line transport, where they would usually be unless there is a prospect of having to bridge a river; if there is work for the engineers, it will be in front and usually, until the engineers have done their work, the column will not be able to advance. It is so easy to get your engineers off the road and out of the way if they are at the front and not wanted there, but it is quite another thing to get them quickly to the front from the rear.

We will now imagine that you have got your sappers to the broken bridge and are fuming to see it repaired. On maneuvers these things are simple, a piece of wood is put over the supposed gap and after a specified time the bridge is judged by the umpires to be repaired. This is a bad lesson. Bridges are not so easily repaired, for remember the spirit may be willing but the material absent; nice mast-like fir trees do not always grow on the spot. I suppose generally it would come to ripping up a floor of some neighboring houses, which might be a little way off. And this is a point I want to make. Earlier in this lecture I asked you to help us with labor, and now I want you to realize that we shall often require transport also. Material will have to be brought in from a distance, planks and joists for bridge repairing, barbed wire, tools, etc., when putting a position in a state of defense, so do not

grudge transport if we ask for it and you can spare it, it will be used in your interest.

You know, of course, that every field company has pontons with it capable of bridging a 25-yard span to carry ordinary wheeled traffic, but it is not every river that is suitable for this class of bridge, there must be depth of water to float the pontons for instance, and then it is not always advisable to tie up your pontons so to speak, I mean use a ponton bridge in preference to repairing a broken bridge, because the loss of your pontons may be serious to you if there is a river to cross later when in touch with the enemy.

I am not ertain that the tactical value of pontons is always appreciated. Two years ago you know the maneuvers hung on the passage of the Thames. I had hoped to see ponton bridges freely used and troops crossing at unforeseen places as a surprise to the enemy (a 25-yard bridge could have been thrown in some fifteen to twenty minutes anywhere), instead of this, it seemed to me that commanding officers preferred to wait until the ordinary masonry bridge had been repaired, forgetting that its position was accurately known to the enemy, who in real warfare would have kept the defile well under artillery fire. Pontons are horsed with six horses in war, so can go across country.

We are now in touch with the enemy, and the dispositions for attack are made, but not until after the reconnaissance. On maneuvers this consists of counting red flags or observing lines of men on the slope of a hill supposed to be hidden away in invisible trenches. Now, of course, Gentlemen, you realize that the real thing is very different. I would just like to quote to you what they think of this abroad. Colonel Polyanski, in the February, 1909, number of the *Injenerni Jurnal*, says:

"Reconnaissance or scouting in the attack is, properly speaking, a duty which belongs to cavalry, but such are the conditions of modern war, that it has become impossible for the cavalry to deal with all questions which have to be answered before the plan of attack can be decided upon. The modern arrangements for the fortification of a position are so complicated and reconnaissance is rendered so difficult by means of masking and the use of dummy works, that none but engineer specialists can understand from long distances the intention and character of the various fieldworks with which the enemy may have added to the strength of his position.

"That special engineer reconnaissance is necessary, first became evident during the Russo-Japanese War. In the month of September, 1904, when attack operations were in contemplation, it was decided to form engineer reconnaissance detachments, and these were recruited from among the officers of the engineer and sapper units."

This is a matter which hitherto has not been much studied. I must confess that I see difficulties under peace conditions, but I

should like you to realize the necessity and try and train your engineers accordingly.

The reconnaissance having been made, the orders for attack are got out. At recent maneuvers it has been the practice to attach a field company to each of the leading brigades, but we must be careful not to adopt this as a hard and fast rule. Engineers must be distributed in accordance with their probable requirements, and I venture to think that in all cases the C. R. E. should be made responsible for that distribution after being made fully acquainted with the conditions, and that, even after distribution, the C. R. E. should still keep in touch with his command and, should circumstances change, advise a redistribution. For instance, the G. O. C. might suddenly decide during an action to have a position prepared to fall back upon, or a series of bridges might be required on a flank, in fact, anything might suddenly arise to make it necessary to withdraw the field companies from the brigades, so brigadiers should remember that at any moment they may be deprived of their engineers. Perhaps some might say "Well, I do not think that will matter much, for I do not see what use they can be to me in the attack." Unfortunately, maneuvers are not quite the real thing, or I venture to think these brigadiers would soon change their tune. Even on maneuvers I have seen a brigadier purple in the face because, through some mistake in orders, his field company had been left with the 2d line transport; the brigadier had got into a village and would have given his eyes for engineers to make loopholes for him, blow down a few inconvenient walls, construct a few barricades, etc., but he had left them behind and got turned out of his village; and I was pleased, for it was a good lesson.

Gentlemen, even if you do not know what to do with your engineers, for goodness sake have them well to the front; you never know when you may want the gun-cotton and crowbars.

I know that in the stress of maneuvers, when everything goes about ten times as fast as it would in war, the unfortunate brigadier is kept at the end of his telephone and has little time to think of the accessories which do not count on maneuvers. I do not know whether you will think me right, but I tell our people it is their own fault if they do not get employed. It should be their business to help the brigadier, and they can best do this by keeping in touch with events. In my opinion, the major commanding the field company temporarily allotted to a brigade should be with the brigadier, and, of his four officers, at least two should be out to the front looking out opportunities for being useful; the artillery are hung up at a boggy place, a few planks and balks would make all the difference; guns can not get to an otherwise favorable position in a wood for want of a road being cut into it; the flank of the artillery position is much exposed, a suitable building exists there which, if put in a state of defence, would make for safety; the colonel of No. 1 Battalion sees his men hard pressed, he wants

suitable rallying points formed, or a bridge blown up to stop the pursuit of the enemy, or a partially destroyed bridge repaired, to enable him to get up his ammunition and machine guns; obstacles removed; captured positions made strong, etc.; in fact, there are innumerable jobs, big and small, where the engineers can be of use. I hope that in our training manual about to be issued, the duties of the R. E. in the attack will be laid down as follows:

1. Assisting the various arms to cross rivers, streams, difficult country, etc.
2. Strengthening ground won, and special points, to help the resistance against a counter-attack, or to serve as pivots of maneuver.
3. The close reconnaissance of an occupied position.
4. Removing or destroying obstacles prior to the final assault.
5. Improving and marking communications.
6. The erection of observatories.
7. Water supply.
8. Fighting when required.

But again let me remind you that unless these things are practiced in peace, they will not be carried out intuitively in war. We want to be so much in touch with you, that you should get all these aids to your advance without your having to ask for them, so to speak.

Perhaps I might here be allowed to make a few remarks as to brigade training. I am sorry to say that I am not myself intimately acquainted with the routine of brigade training, but I imagine that you practice the attack and defense, and that in most cases some R. E. are put at the disposal of the brigadier. Since on maneuvers a field company is almost always attached to each leading brigade, we may assume that the practice will be carried out in war. Brigadiers should remember this and realize that the field company may often be a part of his command. He will then see the advantage of training it and his infantry brigade as one homogeneous whole. We have practically given up moving our field companies from station to station; so far as I understand the policy, the Fifth and Eleventh (Field) Companies will always belong to the Second Division, so you see this homogeneous training I dream of can be made a reality, because the officers, non-commissioned officers and men of the R. E. are the same that will be with the brigade on the battlefield, and it is obvious what a pull the brigadier will have by knowing engineer officers personally and so training them to his ways that they will act intuitively on the battlefield. For brigade training, unless bridges or defenses are to be actually constructed, it is, in my opinion unnecessary to take out the rank and file, since these can be better employed keeping up the knowledge of their trades in workshops than in marching about doing nothing; but all officers, non-commissioned officers, and tool carts, accompanied by a few sappers, should go

out and, in the attack and defense or retreat, the brigadiers should make it their business to think how the R. E. can serve them and issue orders acordingly, just as they would in actual warfare. .The R. E. can then make all their dispositions, get the necessary tools on the spot, arrange for collecting material, appeal for working parties, in fact, do everything except the actual work, and the actual work is an easy matter when a good preliminary plan has been made and tools, material, and personnel collected.

I feel certain that in real warfare, during the attack, for instance a brigadier will have calculated in his mind the chances of a temporary set-back due to counter-attack, etc., and, therefore, the need of rallying points. But this is only one of the innumerable ways in which he can get assistance, and the more he thinks out the problem and uses his imagination from an active service point of view, I am certain he will more and more realize the extreme importance of practicing these matters in peace. Even when he has no use for the men, he should remember that there are six more or less intelligent officers with each field company, each with a horse, and all dying to be his slaves in reconnaissance work, orderly work, or what not.

I shall deal with the defense later, but I have often thought that, when it is a question of a marked enemy, you have an excellent way of instilling into your engineers your ideas as to how to defend a position, and in this way could gain confidence in their work should the time ever come when you have to trust to them to take up and strengthen a position for you in your rear for you to fall back on or otherwise. I suggest to divisional generals that they might give us a chance, now and then, to show our prowess in strengthening a position, by allowing us to work out the marked part, and we might even add a little ''cunning'' and means to deceive, which would add interest. At company training, when fieldworks are being done, I suggest that an officer from the field companies should meet the infantry company officers, and that they discuss the question of the siting of trenches, and their arrangement for mutual support, the siting of obstacles, etc.

Well, so much for the attack.

The defense should, of course, be to the glory of the engineer. But I am not certain that we have yet arrived at the proper way to use him. Perhaps we do not sufficiently discriminate between a deliberately intrenched position and position which a force takes up in the course of a battle and where the men dig themselves in, more or less, where they happen to be, and more to get cover whilst resting or organizing for a further advance than with a view of standing their ground on the position, wearing out the attackers and then going for them and finishing them off in a good counterattack.

In the former there can be but little science; in the latter there should be a very great deal.

In watching maneuvers, I have observed that it is often the prac-

tice to break up a field company into sections (you know a field company consists of four sections, each of some 33 men, 1 tool cart, 1 forage cart, and 1 pack) and distribute these all along the line. Now, personally, I do not think this is an altogether sound arrangement. For maneuver purposes it may be convenient, because you will not have the Inspector of R. E. telling the Inspector-General that the R. E. are not employed, but it is a slovenly, sealed-pattern way of doing things, which does not appeal to me. My view is that, if field companies are allotted to brigades in the front line, the brigadier should take his O. C. Field Company to look over the position, and then decide what is the important work to do. There will be communications to be made in rear, to enable reserves to be gradually brought up, rallying points strengthened, bridges in the front constructed to admit of the further advance, clearances and roads to admit of the artillery getting to good positions, topping distant trees, etc. So I consider that field companies should be held together until it is decided what work there is for them, and not broken up and distributed all along the position on the chance of there being something for them to do.

The course of events may make it desirable to fight an offensive-defensive battle on the position, by which I mean, as I said before, to allow the enemy to wear himself out against the position, and then go for him with all you are worth, to swallow him up in a great counterattack. For this I venture to think a more scientific arrangement can be made than that of merely improving the trenches formed by the troops, when they dug themselves in with a view to further advance. More care must now be taken to coordinate work and add those accessories so essential to a properly defined position. Judging by what I have seen, I am sure that much more care should be taken in coordinating work than has usually been the case. I have seen one battalion commander take the top of the crest, whilst his neighbor took the bottom of the slopes. I have seen trenches from neighboring sections arranged so as to fire into one another, but I have not often seen one section commander arrange his trenches so as to support his neighbor. For proper coordination you want a well-thought-out plan, and I believe it will be well worth your while to cause a roughly contoured map of the position to be made, showing approximately the arrangement of your trenches, etc. Given a 1-inch map, it is astonishing how quickly they can be enlarged to a bigger scale, and with, say, four officers good at sketching, and a mile of front to do between them, viz, 440 yards each, a sufficiently accurate map on a large scale would soon be ready, sufficiently accurate for arranging a coordinated scheme of defence.

You see I have in my mind a position so scientifically defended that a minimum number of men are required for the defence, allowing of a maximum for the knock-down blow, the counter-stroke, and this can not be done in a haphazard, happy-go-lucky way. But I am perhaps wandering from my subject, for it is the

employment of the R. E. that we are discussing. Well, in this deliberate defense, as in all others, I think you should take your senior engineer officer into your confidence, and after explaining the position to him, allow him to make his proposals to you for this apportionment of work between the R. E. and the infantry.

Now as regards the employment of your R. E. when strengthening a position, there is nothing laid down, and rightly so, because we are prepared to undertake anything, or should be, but, generally speaking, I should say that the officer commanding the section would decide where his fire trenches are to be, and so coordinate them as to mutually support one another and neighboring sections; he would leave their construction entirely to the infantry, who, of course, should be equally competent to dig trenches, and better qualified for siting them. Only let me remind you that a battalion commander may site his trenches well from his own selfish point of view, but these very same trenches may, on the other hand, be exceedingly badly sited from the general point of view. I mean he must coordinate. Of course we may have many, many views on the siting of trenches. I remember coming across a trench, not so very long ago, beautifully sited for fire over ground from 800 to 1,200 yards from the trench, but a hopeless position for bringing fire to bear on the ground within 300 yards of the trench. The officer who constructed it said that regulations insisted on a good field of fire, and as it was a question of siting so as to bring fire over all ground within 400 yards of the trench, with a bad field of fire beyond this, or siting so as to get a good field of fire beyond the 400 yards, and much dead ground within this zone, he preferred the latter. Now I may be wrong, but I believe that that trench should have been sited so as to bring every bit of ground within 300 yards of the trench under fire, leaving the ground beyond to be got at by cross-fire from neighboring trenches if possible. It is sometimes forgotten that when you are occupying a crest exposed to the enemy's artillery fire, your heads may be kept under by it until the attackers are within 200 yards of you, so that the very extended field of fire is not always so important as would first appear to be the case. To my mind the important thing is that every bit of ground from the trench as far as possible outwards should be under fire from the trench or flanking trenches; that even at the sacrifice of field of fire the trench should be hidden from the enemy's artillery fire, that command, as a rule, takes too high a place in the scale of importance, that it must be borne in mind that where trenches are placed low down on a slope, it is difficult for the enemy's artillery to support the infantry attacks; with the trenches at the top of the slope this support can be given up to the last moment. With trenches low down, almost at the bottom of the slope, it is easy to hide them, and double line of fire, combined with dummy trenches, becomes possible. The argument against the low-down trench is the difficulty of support, but there is no difficulty in constructing trenches for the supports on either side

of the firing trenches. We have got into the habit of digging our trenches for support well behind the firing trench. I cannot understand the reason. They should be close up to the firing trench, and so save communicating trenches.

Then there is the question of retreat; but we do not fight to retreat. Besides, there are nearly always little subsidiary valleys running in at right angles to the main features which can be easily screened to admit of retirements. Of course, there is the other alternative of making your trenches well back from the crest, holding the crest with a few men to bring fire on lines of approach and mislead the enemy by means of dummy trenches, and flanking the advanced slope with artillery fire, which can often be arranged in a fairly low position. The above, of course, are only my opinions, and they are certainly against practice, for we seem always to get in well-exposed positions half-way down a slope, or on the crest. The fact is, there is this instinct, which it is so difficult to get away from, of being able to see everything from our trenches. Foreign authorities lay stress on the importance of short trenches, not more than 40 yards long, as these can be more easily arranged for mutual support and invisibility than long lines of trenches. For protecting the flanks, they prefer trenches in echelon.

So much for siting trenches. Well, Gentlemen, this is your job, and not that of the R. E., except when forced to do it, as when they are sent to entrench a position on their own, which it will, by the way, often fall to their lot to do, with the assistance of working parties, civilians, Territorials, Special Reservists, and others. No, the infantry should dig their trenches, and I think you will be well advised to employ your engineers on the accessories in laying traps for the attacker, putting cunning and science into the defense, etc. The section commander should, in my opinion, explain to his O. C. R. E. the general scheme of defense, and ask him to suggest the accessories. There are clearings to be made, dummy trenches, and artificial screens to be constructed, entanglements and obstacles cunningly arranged so as to force the attack to come over ground to suit you, dead ground filled in, trees topped which would otherwise block artillery fire, bridges in front of the position destroyed, walls loopholed, communications in the rear constructed, head cover and casemates, in fact, there are a hundred and one things to be thought of beyond the mere entrenchments, and often these are more important than the trenches themselves. Unfortunately, maneuvers do not admit of the use of these accessories being developed, and I fear that in consequence we are not giving them the attention we should do. It would be interesting if some day at divisional training a marked position were specially prepared to admit of these accessories being employed with all the cunning and deceit that the art of the engineers could devise. You would have

to employ special devices to show entanglements, clearing obstacles, dummy trenches, etc., but this would not be difficult.

I expect the attack of such a position would give some fun.

I remember not so long ago a flagged position had been carelessly marked (at least this was the excuse). Two of the red flags had got out of place well to the front of the main line. It was a foggy day. Those few flags completely misled the attack, and it was a long time before the real position was grasped.

I feel we have yet much to learn in this matter of deceiving the enemy.

Now, Gentlemen, before concluding, I should just like to read you a few notes on some foreign ideas on taking up and strengthening a position. It is interesting to know what line others appear to be taking, it provides food for thought.

Well, some of the foreign authorities propose to hold the main line of a position by means of fortified localities or *points d'appui,* occupying those tactical points which the attackers must seize before being able to make further advance.

These *points d'appui* are to be at supporting distance each from the other and arranged, if possible, so as to flank one another.

They are either fortified natural localities, such as woods or villages, or are artificially constructed.

Each is given a definite garrison, which provides its advanced posts, supports, and reserves. The duty of this garrison is to hold on to the ground to the last. The duty of the reserve is to turn out the enemy should he effect an entry into the locality, or deliver offensive returns, and go for his flank should he penetrate the interval between the *point d'appui,* or support a neighboring *point d'appui* should it be hard pressed. In fact, the reserve of each *point d'appui* is, as I understand it, a mobile force for action inside and between *points d'appui* in what is called the "return offensive," but not for action beyond them. The remainder of the garrison allotted to the *point d'appui* is an immobile force strictly on the defensive. The rest of the force provides the grand reserve, which is the mobile maneuver force from the great counter-attack by which alone a victory can be obtained.

A few entrenchments are sometimes of advantage, constructed in the intervals between *points d'appui* to help the reserves of the garrison of the *point d'appui,* or local mobile force in its duty, but these should not be occupied except when the offensive return is being made. The role of the artillery, presumably inferior to the attackers, is chiefly to bring cross fire in front of the *point d'appui* and support the counter-attack. It is said that this arrangement admits of the fewest possible numbers of the defenders being immobilized by occupying trenches, because they appear to consider that troops manning entrenchments are practically immobilized as regards offensive action, and leaves the greater number free for mobile action, and that these can be kept under cover and resting until the moment arrives for that mobile

action. It is also said that by this arrangement the attacker is gradually worn out, that he has to expose his hand and that of the commander, by keeping in communication with the various *points d'appui,* can, as it were, feel the pulse of the attack, and so know where and when to launch his counter-attack, by which alone decisive results can be obtained.

Naturally, these *points d'appui,* whether natural or artificial, must be made very strong, with ample cover for supports close up to, and connected with, the fire trenches. A keep should be provided—something solid and storm proof—to enable some portion of the defenders to hold on till help can be sent, even were the trenches rushed in a night attack, for instance.

Some authorities point out that it is the business of the commander of the *point d'appui* to try and get the attackers to go for him, and not to deter him from doing so by long distance fire, etc., because the mere fact of a serious attack being made means that a force of four or five times that of the garrison is being employed, and so a greater number of the enemy is being immobilized than in the defense; whereas, if the commander, by showing his strength, prematurely frightens off the attacker, it means only that the enemy's forces, which would otherwise have been committed and so immobilized, will go to swell the attackers elsewhere.

The attacking artillery, firing on a *point d'appui* at long range, can not be said to be immobilized, because it has the power of diverting its fire to neighboring *points d'appui;* it can only be said to be immobilized when it is not in its power to do this, that is to say, when it has been obliged by circumstance to come in so close so that all change of objective becomes impossible, or that the infantry that it is supporting requires its undivided attention.

The attacking infantry can only bring pressure on a *point d'appui* by being sufficiently close to it, but this attacking infantry can not necessarily be said to be immobilized, because up to a certain point it is at liberty to break off the attack.

So you see a *point d'appui* is liable to a distant artillery attack, and may have to resist an infantry attack without the pleasure of knowing that the attackers are really committed and, therefore, immobilized as regards actions on other parts of the battlefield, and the commander is not to be congratulated when, by his energetic action, he has warded off an attack.

A *point d'appui,* then, to fulfil its mission, must be ready to receive shot and shell and bullets without replying, and must, therefore, be well organized for defense.

Some authorities believe in taking up the main line of defense some distance behind the main advance crest, that is to say, on a secondary crest, or even on the reverse slope, if not steep. They point out that if on the main crest, which is fully exposed to the enemy's artillery fire, your infantry are subjected for a long time to nerve-shattering effect of an artillery bombardment, and are forced to keep their heads down until the attackers are within 200 yards

of the position, when supports can not reach them without elaborate arrangements in the way of covered communications, whereas if the front crest is occupied by a few well-placed detachments, in trenches, with dummy trenches in between, the power of the rapid smokeless fire of the modern rifle will allow of the enemy being deceived, and he will probably deploy for attack and carry the crest. Once this done, you have them at a disadvantage, because you can sweep these crests and the intervening ground with your artillery, when he can not use his artillery, and the conditions for counter-attack become very favorable.

Well, Gentlemen, these are some foreign ideas; there are many more. There will always be many opinions on this difficult subject. There can be no hard and fast rule for taking up and strengthening a position, but, undoubtedly, you will enormously handicap your adversary if you can manage to take up a position which is not an obvious one to him and not defended in an obvious way to him. If you can do this, you may deceive, confuse, and delay him, and sometimes so manage as to prevent him making effective use of one of his arms, such as his artillery, and at the same time retain full scope for your own, and so bring about all the conditions for a successful counterattack.

Book Review

An Army Officer on Leave in Japan. By Brig. Gen. L. Mervin Maus, U. S. Army. Published by A. C. McClurg & Co., Chicago, Ill.

This book is a singular combination of fact and fancy. It may best be described as "chatty;" it relates the incidents of a trip over Japan by an American Army officer and will, perhaps, arouse a mild interest in anyone who may be contemplating such a trip or who has covered the same routes before.

If we should embellish the prosaic pages of one of Baedeker's guide books with a copious sprinkling of spicy and gossipy anecdotes, add some personal topics of a nature to require them to be ordinarily whispered behind fans, and then garnish with some rambling comments on Eastern religions and politics, the result would be a literary salad not unlike this book.

The photographs are excellent and the style light and readable, such as would pleasantly pass away many an hour in a hammock on a summer day.—W. H. H.

Selected Articles of Engineering Interest

Compiled by Henry E. Haferkorn, Librarian, Engineer School.

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering-Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40 Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution
(52) Barge Canal Bulletin.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.

BLASTING.

Blast which moved 400,000 cubic yards of rock. (15), Feb. 24, 1912. D. I.—Sautage d'une masse compacte de fonte. (47), Feb., 1912.

BREAKWATERS.

Breakwater at Svaneka, Island of Bornholm. (11), March 15, 1912. D. I.—Engineering on the Gold Coast. W. H. Baker. (Cassiers Magazine), March, 1912. I.—How breakwaters are kept in line during construction. H. F. Alexander. (14), Feb. 22, 1912.—Rapid method for estimating the cost of pier and breakwater construction. Liljencrantz. (30), May-June, 1912. I.

CABLEWAYS.

African freight cableway. C. A. Tupper. (14), Feb., 1912. D. I.—Cableway accident at the Pathfinder dam. (14), March 28, 1912. D.—Suspended passenger cableways up Mount Blanc. F. P. Mann. (15), March 30, 1912. I.

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Engineering works at the Rosyth naval dockyard. (11), Feb. 16, 1912.

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COAST EROSION.

Coast protection in Holland. (15), March 9, 1912.

COFFER-DAMS.

Cofferdams and steel sheet piling. E. Wegman and J. R. Wemlinger. (39), March 15, 1912. D.—Construction of cofferdams. (15), March 2, 1912.—General methods of difficult cofferdam construction at Koochicing Falls power development, Minnesota. (12), March 13, 1912. I.—Method of constructing a steel sheet pile cofferdam in 37 feet of water for the removal of the battleship *Maine*. (12), March 27, 1912. D.—Method of repairing a break in a cofferdam. (12), Feb. 28, 1912. D.—Methods of making culvert extensions in widening roadbed, New York Central and Hudson River Railroad. C. E. Anderson. (12), March 6, 1912.—Method of sealing a cofferdam with crushed rock and grout. (12), April 10, 1912.—Placing bracing in a cofferdam before pumping out the water. W. Artingstall. (14), Feb. 22, 1912.—Steel-sheet piking for cofferdams. (11), Feb. 2, 1912. I.

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Dredges and dredging in Mobile Harbor. J. M. Pratt. (12), March 20, 1912. D. —Harbor dredging. B. Cunningham. (Cassiers Magazine), March, 1912. I.—Method of righting an overturned dredge. (12), Feb. 21, 1912.

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Forests and floods on the North Pacific Coast. H. M. Chittenden. (14), April 11,

FOUNDATIONS. (See also, Concrete, Hydroelectric Plants.)

Construction of cofferdams. March 2, 1912. Heavy power house foundations. (15), March 2, 1912. D.

GREAT LAKES.

Use of the Great Lakes. R. M. McCormick. (18), Feb., 1912.—Uses and levels of the Great Lakes. F. G. Ray. C. McD. Townsend. G. S. Williams. (14), Feb. 22, 191.

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Chicago Pacific Railway Co.'s irrigation project in Alberta. (15), March 23, 1912. I.

INLAND NAVIGATION.

Chicago River and the Chicago harbor. (14), March 21, 1912.—Recent Lower Mississippi waterway improvements. C. S. Smith. (30), May-June, 1912. I.

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Engineering on the Gold Coast. (Cassiers Magazine), March, 1912. I.—How to

build a jetty on a sand bottom in an open sea. J. F. LeBarron. Discussion. (21), Feb., 1912.—Reinforced concrete jetties in England. (7), March, 1912. I.

LAND RECLAMATION.

Description of the Prosser division of the Dunnyside unit. Yakima project. U. S. Reclamation service. E. A. Moritz. (14), March 28, 1912. D. I.—Ditch system for draining 103,000 acres in Washington Co., Miss. (12), April 3, 1912. D.—Land reclamation by flood. (12), Feb. 28, 1912. D.—A sandy belt on the Madras coast. R. Ryves. (10), Feb. 2, 1912. D. I.—Sixth ward and Crowley drainage district, La. W. S. White. (12), March 13, 1912. D.—Some examples of tidal marsh land reclamation; structures and cost. IV. (12), Feb. 21, 1912. D.—Under-drainage of alluvial lands. J. A. Harmman. (15), Feb. 24, 1912.

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Borrow pits in levee building. W. L. Marshall. (14), Jan. 11, 1912.—High water damages due to levee construction. Decision, U. S Supreme Court. (30), May-June, 1912.—Land-side or water-side borrow pits in levee building; comments prompted by the Colorado River problem. A. L. Dabney, E. L. Chamberlain, A. T. Parsons. (14), Feb. 1, 1912.—More about the location of borrow pits in levee construction. F. L. Sellew. (14), April 11, 1912.

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Concrete lock in Scotland. (15), March 9, 1912.—Control of lock machinery. (41), Feb. 7, 1912.—Coverplates in locks. (41), Feb. 14, 1912. D.—Economical lay-out of a concrete plant. G. C. Scherer. (14), Feb. 22, 1912. D. I.—Engineering works at the Rosyth naval dockyard. (11), Feb. 16, 1912.—Last stages of the Panama Canal. (10), Feb. 9, 1912. D.—Lock control systems of the Panama Canal. (14), March 21, 1912.—Lock working model. (41), March 13, 1912.—Mechanical appliances on the Panama Canal. J. F. Springer. (Cassiers Magazine), March, 1912. I.—Methods and cost of constructing Lock No. 6, and guard gate, N. Y. State barge canal. E. J. Becker. (12), March 6, 1912. I.—Methods and costs of construction on contract 11, N. Y. State barge canal. E. J. Becker. (12), Feb. 21, 1912. I.—Operation of the locks of the Panama Canal. (15), March 2, 1912. D.—Tracks of lock towing system. (41), March 6, 1912. D.—Water-power development of the Mississippi River Power Co. at Keokuk, Ia. H. L. Cooper. (18), March, 1912. D.

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Development and tactics of the military bridge equipage. C. A. F. Flagler. (30), May-June, 1912.—Handling our ponton equipage. J. J. Loving. (30), May-June, 1912. I.

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Improving the Upper Mississippi River. E. F. Linderman. (14), April 11, 1912. D. I.

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Alleged volcano in the Culebra Cut. (1), March 21, 1912. Character of reported volcanic heat in Culebra Cut, Panama Canal. (12), March 27, 1912.—Fixing the tolls on the Panama Canal. (15), March 2, 1912.—Last stages of the Panama Canal construction. (10), Feb. 9, 23, 1912. D.—Lock control systems of the Panama Canal. (14), March 21, 1912.—Lock working model. (41), March 13, 1912.—Mechanical appliances on the Panama Canal. J. F. Springer. (Cassiers Magazine), March, 1912. I.—Notes on the Panama Canal. R. A. Owen. (32), April, 1912. D.—Operation of the locks of the Panama Canal. (15), March 2, 1912. D.—Panama Canal dues and the U. S. merchant marine. (10), Feb. 16, 1912.—The question of canal tolls. (27), Feb. 17, 1912.—Tracks of lock towing system. (41), March 6, 1912. D.

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Rapid method for estimating the cost of pier and breakwater construction. Liljencrantz. (30), May-June, 1912. I.

PILE DRIVERS AND PILE DRIVING.

Construction of Desplaines Valley Railroad. (39), March 15, 1912. I.—Locomotive steam pile-driving plant. (10), March 22, 1912. D. I.—Methods of making culvert extensions in widening roadbed, New York Central and Hudson River Railroad. C. E. Anderson. (12), March 6, 1912.—Reinforced-concrete viaduct carrying a Seattle street over railway yards. E. E. Adams. (14), March 21, 1912. D. I.—Technical considerations in the designing, molding, and driving of reinforced concrete

piles. (12), April 3, 1912.—White River development of the Pacific Coast Power Co. (15), April 13, 1912. D. I.

PILES AND PILING. (See also, Concrete.)

Cofferdams and steel sheet piling. E. Wegman and J. R. Wembinger. (39), March 15, 1912. D.—New pile formula. (15), March 2, 1912.—Steel screw piles. (15), Feb. 17, 1912.—Steel sheet piles for cofferdams. (11), Feb. 2, 1912. I.

POLLUTION OF RIVERS.

Acid conditions in the Monongahela River. (23), March, 1912.—Enclosing a sewage laden stream in a triple-barreled concrete conduit. (15), March 16, 1912. D. I.—River boards for sanitary and general control. R. Hering. (14), Feb. 22, 1912.

RESERVOIRS. (See also, Dams, River Regulation.)

Cost of constructing a small reinforced concrete reservoir. C. A. Bingham. (12), April 3, 1912. I.—Methods and cost of lining a brick-lined reservoir with concrete. (12), April 3, 1912. D. I.—Proposed reservoir system on the Allegheny and Monongahela rivers. (14), April 4, 1912.

RETAINING WALLS. (See also, Concrete.)

Graphic determination of pressure on retaining walls. J. A. Main. (11), March 1, 1912. D.—Method and cost of constructing a reinforced concrete retaining wall with rear anchorage. R. A. Bootle. (12), March 20, 1912. D.—Standard practice in reinforced concrete highway bridges and culverts in Iowa. (12), March 13, 1912. D.

RIVER ENGINEERING. (See also, Erosion.)

Borrow pits in levee building. W. L. Marshall. (14), Jan. 11, 1912.—Borrow-pit practice on the Yuma project, Lower Colorado, and testimony in favor of riverside pits. L. L. Sellew. (14), Feb. 15, 1912. D.—Causes of river deviation. E. F. T. Bennett. (10), Feb 9, 1912. D.—Improving Upper Mississippi River. E. F. Linderman. (14) April 11, 1912. D. I.—Land-side or water-side borrow pits in levee building; comments prompted by the Colorado River problem. A. L. Dabney, E. L. Chamberlain, A. T. Parsons. (14), Feb. 1, 1912.—More about the location of borrow pits in levee construction. F. L. Sellew. (14), April 11, 1912.—Proposed reservoir system on the Allegheny and Monongahela rivers. (14), April 4, 1912.

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Project d'amenagement de la riviere La Dranse au moyen de trois chutes successives. E. Pacoret. (48), March 1, 1912. D.

ROCK EXCAVATION.

Blast which moved 400,000 cubic yards of rock. (15), Feb. 24, 1912. D. I.

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The *Maine* raised. (15), March 16, 1912. I.—Salving of the submarine boat A3. (11), Feb. 16, 1912.

SHIP-CANALS.

Manchester ship canal. (11), Feb. 23, 1912.

SURVEYS AND SURVEYING.

Mule-back reconnaissance. Discussion. A. J. Millard. (21), March, 1912.—Retracement-resurveys. N. B. Sweitzer. (21), Feb.-March, 1912.

TOWING. (Canals.)

Panama Canal electric towing locomotives. (14), March 21, 1912. D.

WATER POWER. (See also, Hydroelectric Plants.)

Contraction of water power control. H. K. Smith. (14), March 28, 1912.—The value of water power. (15), Feb. 17, 1912.—Water power development. (49), April 13, 1912.—Power development at the Falls of the Ohio River. L. Brown. (30), May-June, 1912. I.

WATERPROOFING.

Inspection of waterproofing for concrete work. J. Cochran. (12), April 10, 1912.—Methods of testing waterproofing compounds. (40), April, 1912.—Reinforced-concrete viaduct carrying a Seattle street over railroad yards. E. E. Adams. (14), March 21, 1912. D. I.—Waterproofing engineering structures. (15), April 6, 1912.

WATER TERMINALS.

Chicago River and the Chicago Harbor. (14), March 21, 1912.

WHARVES.

Lumber wharf at Balboa. (41), Feb. 7, 1912.—Reinforced concrete wharves. (15), Feb. 24, 1912.

Editorial Notes.

The recent floods in the Lower Mississippi have exceeded in heights the levels feared as possible, though not probable, by those interested in levee construction. Previous to this year the greatest flood on record was that of 1883, when the gage reading at Cairo was 52.17. At points below Cairo the gage readings of the 1883 flood have been exceeded many times, on account of the new conditions introduced by the construction of levees. These conditions have varied from year to year as levee construction has progressed, making comparative gage readings at points below Cairo of no value, except locally, for use in determining the height to which levees in that vicinity should be built. Although during the recent flood the river reached a stage of 54 feet on the Cairo gage on April 6th and 7th, the resulting stages in the lower river, while greater than any heretofore recorded, were undoubtedly lower in many stretches than they would have been had the levees not been overtopped in places. Through the crevasses thus made in the levees an enormous amount of water was diverted from the river, and this undoubtedly accounts for the fact that the highest gage reading at Memphis (44.8 feet) occurred on the same day the crest reached Cairo, 230 miles above. The gage reading at Memphis had fallen 0.3 of a foot on the following day, though the crest at Cairo remained at its maximum height of 54 feet. The following table shows the maximum stages of the principal tributaries contributing to the recent flood in the Lower Mississippi.†

Distance from Cairo.	Gage.	River.	High water 1912*	Date.	Previous max. high water	Date.
510	Rock Island	Upper Mississippi	12.7	3-30	19.4	1892
985	Sioux City	Missouri	13.1	3-29	22.2	1881
498	Cincinnati	Ohio	53.2	3-27	71.06	1884
248	Nashville	Cumberland	37.9	3-19		
			45.8	4-6	55.3	1882
509	Chattanooga	Tennessee	23.9	3-17		
			31.3	3-31	58.6	1867
0	Cairo		54.0	4-6	52.17	1883
220	Memphis	Lower Mississippi	44.8	4-6	40.3	1907

*To include April 6, 1912, only.

† Data furnished by United States Weather Bureau.

A study of this table shows that while none of the contributing rivers reached a stage even approximately equal to their previous high waters, all were at a gage about two-thirds of the maximum, and when we consider that the increased flow due to the upper third of the gage reading is greater than the middle third and very much greater than the lower third, we see that the greatest recorded flood in the Lower Mississippi Valley is due, not to the maximum flood of any one river, but to an unfortunate combination of medium floods in its various tributaries. The flood of 1883 came largely from the Ohio Basin, where the annual flood is due about March 1st; whereas, the floods from the Upper Mississippi and Missouri Rivers do not ordinarily come until May or June, giving rise to the well-known term the "June Rise."

Due to the Upper Mississippi being frozen over, there were no gage readings previous to March 30th, 1912, but since that date the river has fallen almost continuously up to the present time. The maximum stage at Sioux City occurred only nine days before that at Cairo, so that the Upper Missouri did not contribute its maximum flood to the highest stage in the Lower Mississippi.

The flood contributions are difficult to analyze, because the main crests of the floods on the Cumberland and the Tennessee could hardly reach Cairo in time to contribute to the maximum stage at that point. The culmination of the flood at both Cairo and Memphis on the same day is also unusual, as it generally takes from seven to eight days for the crest of a flood to pass from Cairo to Memphis, the crest travelling at the rate of about 30 miles a day. It is easy to explain this, however, by the breaks in the Lower St. Francis levees, which allowed enough water to flow through the crevasses on the Arkansas side to keep the gage readings at Memphis lower than they would have been had no breaks occurred.

No detailed reports of the flood have yet been made, as it is still in progress on the lower river; but as this flood is the greatest known throughout a period of half a century, the data in regard to it and the information to be obtained therefrom will be awaited with interest, not only by the engineers engaged in levee construction and by the inhabitants of the leveed districts, but also by the public at large.

The report of the Board of Assistant Engineers on the award of the prize for the best article appearing in Volume III of the

MEMOIRS has just been received. The Board awards the prize to Mr. J. S. Walker, Assistant Engineer in the United States Engineer Office, Nashville, Tenn., and its report is as follows:

THE BOARD OF EDITORS,
PROFESSIONAL MEMOIRS,
Washington Barracks, D. C.

GENTLEMEN: The undersigned, acting as judges in the selection of the best article by a civilian assistant of the Corps of Engineers, published in Volume III of the PROFESSIONAL MEMOIRS, beg leave to recommend award of the prize offered, to Mr. J. S. Walker, Assistant Engineer, for the article appearing in the October-December, 1911, number under the title "Elements Affecting Lock Construction on Canalized Rivers Having Fixed Dams."

Very respectfully,

EDMUND MOESER,
Assistant Engineer.
D. M. ANDREWS,
Assistant Engineer.
H. C. GOULD,
Assistant Engineer.

April 22, 1912.

We desire to thank the members of the Board for acting as judges, and to extend our congratulations to Mr. Walker in winning the first prize offered by the MEMOIRS.

In connection therewith we desire to call attention to the offer in No. 13, page 154, in which the number and value of the prizes have been considerably increased, and we hope that during the rest of the year we shall receive many papers in competition for these prizes. It is not believed that the money value of the prizes offered will stimulate writers so much as the recognition of superiority in the papers to which the prizes are awarded. The MEMOIRS is not financially able to pay liberally for articles contributed, and so must in a large measure depend upon the esprit de corps of its readers for the subject matter of the magazine.

The competitive examination for entrance to the Corps of Engineers was taken by four men only, of whom one, Mr. Wistar Morris Chubb, of Ohio, passed successfully and has been commissioned a Probational Second Lieutenant in the Corps of Engineers for a period of one year.

It seems rather surprising that only four men should take the examination. The *Engineering News* says, "The examinations as advertised were not too difficult for graduates of a reputable

engineering school, so it appears that the reason must be sought elsewhere.'' Inasmuch as each competitor for the final examination must be eligible for employment as a Junior Engineer in the Engineer Department, it may have been that the time available for passing the Junior Engineers' examination and preparing for the second examination was so short as to discourage competitors. This can not be the case in the future, since a man can take the examination for Junior Engineer at such time as to give him plenty of time for preparation for the final examination. Quoting from the *Engineering News* again, ''It appears to us that to be an officer in the Engineer Corps of the U. S. Army is an honorable estate well worth the efforts of any young engineer * * * ; the lack of interest in the examination this year is no fair measure of what may be expected in the future. * * * ''

Change of Address

The addresses of subscribers will be changed as often as desired, and will be made up to one week before the magazine is issued. We desire to say for the benefit of those subscribers who are officers in the Army, that while every effort is made to keep addresses changed up to date in accordance with official orders, yet it is found impossible to do so satisfactorily unless each subscriber notifies the magazine of the time he desires his address changed. The reason for this will be readily appreciated when one considers that orders transferring officers from the Philippine Islands to the United States, and vice versa, are issued frequently several months before they are to take effect and, occasionally, as has happened during the past year, these orders are revoked or their execution is delayed. Again, many officers take advantage of leaves of absence of from two to four months, so that the dates of arrival at new stations are impossible of computation.

Accordingly it is earnestly requested that all subscribers, including those whose addresses are changed by official orders, will notify the Professional Memoirs of their new addresses and the dates they wish them to go into effect—remembering that the Memoirs is issued on the first day of each odd-numbered month.

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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Contents

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BRIG. GEN. THOMAS LINCOLN CASEY

CHIEF OF ENGINEERS, UNITED STATES ARMY
1888-1895

BORN 1831—DIED 1896

The Plaquemine Lock

BY

Capt. ROBERT R. RALSTON
Corps of Engineers

To properly understand the reasons which prompted the improvement of Bayou Plaquemine, a short description of the waterways of that part of southern Louisiana in the alluvial valley of the Mississippi River might not be out of place.

An examination of a map of southern Louisiana will show the Atchafalaya River branching off from the Red River at Turnbulls Island, about 210 miles above New Orleans, and a few miles above the junction of the Red and Mississippi rivers, as the former joins the Mississippi at this place by what is known as Old River.

The Atchafalaya, after flowing almost due south for 67 miles as one stream, breaks up at a point called Butte La Rose, into a number of smaller streams which merge into Grand Lake a short distance farther south, except the one known as Grand River, which sweeps to the eastward of Grand Lake and joins it only near its southern end, where all the waters of the Atchafalaya and its tributaries unite to form the Lower Atchafalaya River, emptying into the Gulf of Mexico through Atchafalaya Bay.

Joining the Lower Atchafalaya from the west is Bayou Teche, one of the most important waterways of this section. It penetrates a rich sugar raising territory and is largely used as a means of transportation of agricultural products and other commodities. Several other tributaries of the Atchafalaya tap rich agricultural sections and extensive areas of timber land, and before the advent of the railroad these waterways were the only available means of transportation.

New Orleans was then, as it is to-day, the objective point of a large portion of this commerce, and to reach the Mississippi from the Atchafalaya one of the routes, previous to the Civil War, was by way of Bayou Plaquemine, a small stream connecting Grand River with the Mississippi.

This stream, or bayou, joins the Mississippi at the town of

Plaquemine, La., about 110 miles above New Orleans. Its length is about 11 miles, and it flows in a general south-westerly direction.

According to such information as can be obtained, it could only be navigated during high water, for during the low water season the bayou, near its junction with the main river, was very shoal, due to the large deposits of sediment at that point.

Levees had been built along its banks connecting with the levee along the Mississippi, and the land adjacent to it was under cultivation.

Some attempts were made, it is understood, to improve the bayou so as to permit navigation at all stages of the river, but these attempts were unsuccessful, and in 1867 or 1868 the local officials, in order to render the adjacent territory safe from overflow and at the same time to lessen the burden of levee maintenance by eliminating those along Bayou Plaquemine, closed the bayou near its head by extending the Mississippi River levee across it.

The closure of Bayou Plaquemine left the navigation interests along the Atchafalaya the option of reaching New Orleans by the open Gulf or by going up to the Red River and thence out into the Mississippi at Turnbulls Island by way of Old River, a route 180 miles longer than by way of Bayou Plaquemine.

In the meanwhile, changes were taking place in the Upper Atchafalaya and in Old River which had an important bearing on the question of the reopening of navigation through Bayou Plaquemine. The Upper Atchafalaya, previous to the Civil War, was rather an insignificant stream and was choked near its head by an immense area of fallen timber and drift, more or less covered with sediment, and known as a raft.

In order to improve the upper portions of the stream for navigation purposes the State of Louisiana had, just previous to the war, removed a quantity of this raft to secure a navigable channel to Red River. This permitted a greater flow down the channel during high water in the Mississippi, and as the distance to the Gulf by way of the Atchafalaya is only about one-half as great as by the main river, the swift current caused so much erosion during each high water season that by 1880 the Atchafalaya had enlarged enormously, and at that time was estimated to carry off one-sixth of the high water flow of the Mississippi.

During low water a great part of the flow of Red River passed down the Atchafalaya, and Old River during the low water season

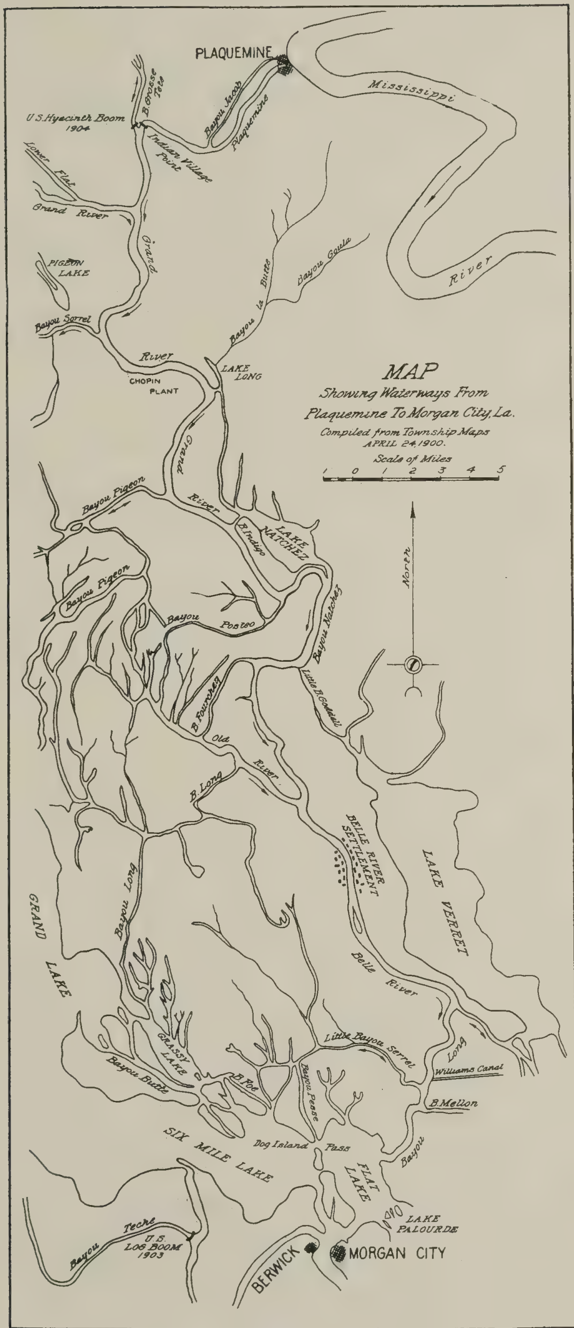

MAP
Showing Waterways From
Plaquemine To Morgan City, La.
Compiled from Township Maps
APRIL 24, 1900.
Scale of Miles
1 0 1 2 3 4 5
North
PLAQUEMINE
Mississippi
River
U.S. Hyacinth Boom
1904
Bayou Jacob
Plaquemine
Indian Village
Point
Lower Flat
Grand River
Grand
PIGEON
LAKE
Bayou Sorrel
River
CHOPIN
PLANT
Bayou la Butte
Bayou Goula
LAKE
LONG
Grand
River
Bayou Pigeon
Bayou Pigeon
B. Indigo
LAKE
NATCHEZ
Bayou Natchez
Bayou
Bayou Fourchen
Little B. Goddell
Old
River
B. Long
BELLE RIVER
SETTLEMENT
LAKE VERRET
GRAND
LAKE
Bayou Long
Belle
River
GRASSY
LAKE
Little Bayou Serrel
Long
Williams Canal
Bayou Butte
B. Poe
Bayou Pease
B. Mellon
SIX MILE LAKE
Dog Island
Pass
FLAT
LAKE
Bayou
LAKE
PALOURDE
Bayou
Teche
U.S.
LOG BOOM
1903
BERWICK
MORGAN CITY

Fig. 1.

began to resemble Bayou Plaquemine before its closure, in that it became so shoal that navigation was practically suspended.

One of the first problems which the Mississippi River Commission took up after its creation, was that of stopping or regulating the rapidly increasing high water discharge of the Mississippi down the Atchafalaya, and also maintaining navigation through Old River during the low water season.

In the problem of the Atchafalaya numerous plans were proposed, among which one was the separation of the Red and Atchafalaya from the Mississippi, and another the separation of the Atchafalaya from the Red.

In either case a new outlet into the Mississippi for navigation interests would be required, but whether the prospect of this separation, the difficulty of navigation in Old River during low water, or the desire for a shorter inland route to New Orleans was the reason having the greatest weight in the agitation for the reopening of Bayou Plaquemine, it is difficult at this time to determine.

As it turned out, the solution of the Atchafalaya problem, for the time being, was found in the use of submerged brush and stone dams at its head to prevent further enlargement, and in the maintenance of a channel in Old River by dredging.

With the recent opening of the route by way of Plaquemine, the question of separating the Red and the Atchafalaya from the Mississippi is again coming forward, but this time it is for the purpose of reclaiming land once cultivated along the Atchafalaya, though now overflowed during high water seasons.

The Act of Congress, June 14, 1880, provided for a survey of Bayou Plaquemine, and from this survey the bayou was found to be very much obstructed for the first 5 miles below the dike or levee closing it, and so shoal as to be entirely dry in places at low water. The principal obstructions were cypress stumps and saw mill refuse from the mills along the stream.

From the 5th to the 7th mile it was obstructed a good deal with saw-mill refuse and sunken logs, but below the 7th mile it was of good width and depth, though with some obstructions in the way of fallen and overhanging trees. The depth of the bayou below the 7th mile was 10 feet and over, and this depth could be carried down Grand River to Morgan City on the Lower Atchafalaya except in two wide reaches of Grand River, known as Bay Natchez and Flat

Lake, where from 4 to 4½ feet at low water was all that could be carried.

Grand River at this time was much obstructed with snags, logs, fallen and overhanging trees, as were the other waterways in the vicinity of Grand Lake, since very little, if any, work had as yet been done on them, with the exception of Bayou Teche.

The Act of Congress, August 2, 1882, in appropriating funds for the improvement of the Mississippi under the River Commission, mentioned among other items ''a lock at Bayou Plaquemine.''

In accordance with a resolution of the Commission, a survey of a navigable route from the Missssissippi through Bayou Plaquemine, and up Grand River and the Atchafalaya to the head of the latter stream, so as to replace the, then, access to Red River, was begun by Major Stickney, in charge of the Fourth District of the Mississippi River.

Based on the surveys thus made, and available maps, estimates were prepared for a waterway 5 feet deep at low water, and with a width of 150 feet in Bayou Plaquemine, and 200 feet in Grand River, for a double lock of brick to make the connection with the Mississippi at Plaquemine, and for the removal of snags from the Atchafalaya. The estimated cost of the waterway, the locks and the removal of snags was $1,708,250.

The Act of July 5, 1884, directed another examination of Bayou Plaquemine which was reported on adversely, on the ground that the bayou was not worthy of improvement, unless the question of constructing locks was considered, and locks were not mentioned in the Act.

The Act of August 5, 1886, directed the examination of the mouth of Bayou Plaquemine with a view to its connection with the Mississippi River by means of locks, also Bayou Plaquemine and other connecting streams to form the best route to Grand Lake.

The examination was made by Major Heuer, then in charge of the United States Engineer Office in New Orleans, and resulted in a favorable report, and his estimate of the cost of the proposed improvement, including the protection of the bank of the Mississippi River near Plaquemine, was the same as that of Major Stickney, $1,708,250.00.

His report, which gives in some detail the necessity for the improvement and the results of the investigations at the site of the proposed locks, both as to character of foundation and necessity

for bank protection, is printed in the Annual Report of the Chief of Engineers for 1887.

In its report for 1887, the Mississippi River Commission recommended that certain funds be allotted from the appropriations for the Red and Atchafalaya rivers, for the improvement of Bayou Plaquemine as far as the dike closing the bayou, so as to afford relief to commerce on the Atchafalaya and its tributaries during the low water season when the outlet through Old River could not be navigated.

By so improving Bayou Plaquemine, it would be possible to transfer freight to boats on the Mississippi at Plaquemine and by such means a certain amount of relief could be afforded.

The Act of August 11, 1888, appropriated $100,000.00 for securing a navigable channel 60 feet wide and 6 feet deep in Bayou Plaquemine from deep water to the Plaquemine dike and for protecting the Mississippi River bank near the mouth of the bayou against further caving.

Of this first appropriation $75,000.00 was allotted to the work of bank protection, and by suitable arrangement transferred to the Fourth District of the Mississippi River, since that district was already in possession of plant suitable for such work. With the balance of the appropriation a United States dredge began work on the excavation of the proposed channel on June 4, 1889, at Dardens Bend, about 5 miles below the head of the bayou.

After this date work on the Plaquemine improvement may be divided into three parts, which will be considered separately, namely:

First. Improving Bayou Plaquemine and connecting streams;
Second. Bank protection near mouth of bayou;
Third. The Plaquemine Lock.

IMPROVING BAYOU PLAQUEMINE AND CONNECTING STREAMS.

The work of improving Bayou Plaquemine continued as funds became available, during 1890, 1891, and 1892, and the work was carried to the Texas and Pacific Railroad bridge at the town of Plaquemine. This was a fixed bridge at that time, but was replaced by a draw in 1892-93 and in 1894 the channel was completed to the dike as originally contemplated.

Appropriations were made in 1890 of $100,000.00, of which $40,000.00 was alloted to the bayou improvement, and the balance to bank protection.

In 1892 $150,000 was appropriated and in 1894 $110,000.00 of which $72,000.00 was alloted to the work of bank protection from the 1892 appropriation, and both appropriation acts provided that at the discretion of the Secretary of War, $10,000.00 of each appropriation might be used in removing obstructions from Grand River and Pigeon Bayous forming part of the Plaquemine Route.

The authorization of the expenditure of these funds on Grand River and Pigeon Bayous being for the purpose of opening up a navigable waterway from the Lower Atchafalaya and Bayou Teche in accordance with the report of Major Heuer, rather than to the head of the Atchafalaya as reported on by Major Stickney, makes it evident that the demand for a shorter outlet to the Mississippi than that afforded by Old River was the consideration at this time prompting the Plaquemine improvement.

The Pigeon Bayous, whose improvement was contemplated by the Acts of 1892 and 1894, consist of Pigeon Bayou and its outlet tributary, Little Pigeon Bayou, and connect Grand River with Grand Lake, Pigeon Bayou leaving Grand River about 15 miles below the mouth of Bayou Plaquemine.

With the funds appropriated and with $10,000.00 additional, which became available in the appropriation of 1896, work was carried on in Grand River from near the mouth of Bayou Plaquemine to near Morgan City and Pigeon Bayous from Grand River to Grand Lake. This work was carried on from 1893 to 1897 as funds became available, and consisted in the removal of snags, logs, fallen and overhanging trees and raft and the dredging of some shoals.

The smallness of the appropriation for these streams necessitated going over the same routes several times, as it was impracticable with but $10,000.00 every two years to do more at first than remove the worst obstructions, and moreover obstructions continued to form from year to year.

The River and Harbor Act of 1896 in making appropriations for the Plaquemine improvement authorized continuing contracts to be entered into, not to exceed $1,173,250.00 exclusive of the amounts theretofore appropriated, and thereby permitted definite plans to be prepared for the continuance of the improvements.

The original project was accordingly modified in April, 1899, so as to provide for a channel in Bayou Plaquemine from the lock to Dardens Bend, about 5 miles, having a bottom width of 95 feet,

slopes of $1\frac{1}{2}$ on 1 and a depth at low water of 10 feet, and involving the excavation of 1,108,000 cubic yards of material.

The modified project also provided for making a cut-off at Dardens Bend, so as to avoid the bend, as the bayou turned about 160° at that place. The new project also contemplated the further improvement of Grand River.

Contract was let for the improvement of Bayou Plaquemine with Charles Clarke & Co., of Galveston, Texas, at 13.98 cents per cubic yard, and work began in October, 1899, the cut-off at Dardens Bend being first completed, and the work then continued upstream toward the lock.

Some work in Bayou Plaquemine and Grand River was done in 1900 and 1901 by a hired dredge, and a survey of Bay Natchez and Flat Lake, two wide and shallow reaches of Grand River, was also made in order to prepare plans for a 10-foot channel through these sections of the river.

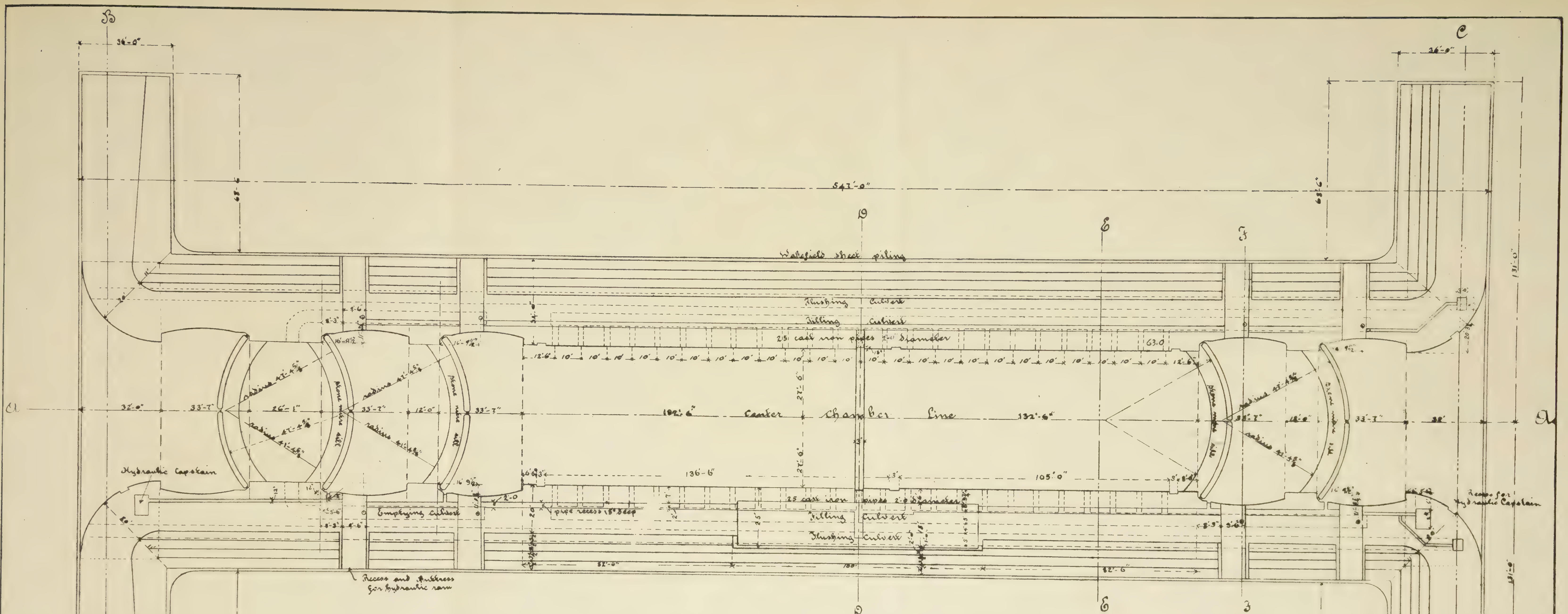
Contract for this work was entered into with Charles Clarke & Co., the contractors for the improvement of Bayou Plaquemine, at 20 cents per cubic yard, and channels 50 feet wide and 10 feet deep were completed in 1903 and 1904. These channels are entirely too narrow for easy navigation, but at the time they were dredged sufficient funds were not available for excavating wider channels.

The contractors experienced great difficulty in the execution of this work in Flat Lake and Bay Natchez, due to the narrowness of the excavated channels, the shallow character of these sections, and the great number of submerged stumps encountered. In the work on Flat Lake dump scows were used, necessitating the excavation of a scow channel alongside of the main channel, the material being dumped in deep water in Berwick Bay, an enlargement of the Lower Atchafalaya.

In Bay Natchez the material was required to be deposited 300 feet from the edge of the cut and the contractors began work with an hydraulic dredge, but this was greatly interfered with by the submerged stumps encountered.

As soon as high water in the Atchafalaya raised the water level sufficiently to float dump scows, the hydraulic dredge was dispensed with, and afterwards during low water the contractors arranged an inclined chute on scows, into which the dredged material was dumped, and by means of a 6-inch stream of water pumped into the chute was carried to the required distance quite satisfactorily.

The contractor for the enlargement of Bayou Plaquemine was



Improving Bayou Plaquemine

Plan of Lock

Made under direction of Major James B. Quinn Corps of Eng'rs U. S. A.
F. H. Thomson Asst Eng'r

Lock located at Plaquemine La.

also experiencing difficulties in that work and his contract, which should have been finished in 1901, was not more than one-half completed in 1905. The contract had provided for dumping the excavated material in Dardens Bend, which had been eliminated from the bayou by the cut-off across the point, but after about 2 miles of the channel above Dardens Bend had been completed the contractor found that the limit of the capacity of the bend had been reached, unless very expensive methods of rehandling the material were provided, and the process of dumping, whatever method was adopted, would be slow and difficult. The work of dredging was also greatly interfered with on account of the many submerged stumps and logs encountered.

The contract was accordingly modified in March, 1905, so as to permit the contractor to delay work on his contract till the lock was in operation, when the dredged material could be taken in scows through the lock and dumped in deep water in the Mississippi River. Work was accordingly suspended in June, 1905, and no work was done till May, 1909, by which time the lock was in operation, and work on the contract was resumed. There is now a waterway with a 10-foot channel from the Mississippi River to Morgan City on the Lower Atchafalaya, by way of Bayou Plaquemine and Grand River.

The waterway is being used at the present time, though some difficulty is being experienced during low water in the portions of Bayou Plaquemine where the dredging has not yet been completed, and in the narrow channels through Bay Natchez and Flat Lake. These latter channels should be double their present width for easy navigation, and it is likely that their enlargement will be taken up in the near future.

Under the original project for a channel in Bayou Plaquemine 60 feet wide and 6 feet deep to the Plaquemine dike, there was removed from the bayou about 356,000 cubic yards of material and some 2,100 snags, logs, stumps, etc., many of which it was necessary to break up by means of explosives.

The revised project contemplates the removal of about 1,108,000 cubic yards which will, when completed, make a total of 1,464,000 cubic yards of material removed.

In the various works of improvement on Grand River and Pigeon Bayous there have been removed to date approximately 560,000 cubic yards of material, 15,800 logs, snags, stumps, etc., and 4½ miles of raft.

BANK PROTECTION NEAR THE MOUTH OF THE BAYOU.

This work, as previously stated, was placed under the Fourth District of the Mississippi River, and a project was prepared and approved for the construction of four large spur dikes, two above and two below the mouth of the bayou, spaced about 1,000 feet apart.

It was expected, no doubt, that the lock would be so situated that the mouth of the bayou would form the river approach, which would seem to explain the location of the proposed spur dikes, but later the river approach to the lock was located about 1,000 feet below the mouth of the bayou between spurs Nos. 3 and 4, and a fifth spur was added to the project and placed below No. 4.

Work was begun on the spur dikes on November 20, 1889, using the plant from the Red and Atchafalaya rivers, but a good deal of delay had occurred, due to the long distance it was necessary to transport the materials, principally the rock, used in construction.

Owing to the late date on which the work was begun, it was only possible to complete spurs Nos. 1 and 2 above the mouth of the bayou when high water, in January, 1890, caused a suspension of the work.

These spurs consist of a foundation mattress, with its shore end about low water mark, and tiers of cribs placed one upon another and loaded with rock or gravel in sacks.

The foundation mattress of each spur was built up of layers of willow brush at right angles to each other, firmly compressed and clamped by means of sawed scantlings, in pairs, so as to be about 2 feet thick.

The cribs were similar to the mattresses in construction, but were 6 feet thick, and heavier scantling was used in clamping.

Spur No. 1 was 510 feet long and 20 feet high, with a sill mat 100 feet wide.

Spur No. 2 was 330 feet long and 32 feet high, with a sill mat 110 feet wide.

Work was resumed in September, 1891, when the river was sufficiently low, and by December of that year spurs Nos. 3 and 4 and the additional spur, No. 5, were completed and some minor repairs made to No. 1.

With the completion of these five spurs, the bank protection extended for 2,500 feet above and 1,500 feet below the proposed entrance to the lock. The cost of this work was about $103,000.00.

Caving continued above and below the dikes, and eventually

some caves developed between the spurs, which made it apparent that the spur dikes alone were not sufficient to prevent caving.

It was then proposed to supplement the dikes by placing mattresses between them so as to form a continuous revetment, omitting the space between dikes Nos. 3 and 4, where the approach was to be located, until the lock was completed.

The work was still under the Fourth District of the Mississippi River, and during the low water season of 1893-1894 1,200 feet of continuous revetment was placed above the approach to the lock and 1,400 feet below, omitting about 500 feet opposite the river approach, at a cost of about $103,000.00 additional.

This work consisted of placing brush and pole mattresses loaded with rock between the spur dikes, the mattresses being 150 by 400 feet and about 2½ feet thick.

With the completion of the continuous revetment work, the work of bank protection was again placed under the office in charge of the Plaquemine improvement, on September 28, 1894.

Caving took place near the mouth of the bayou in September, 1895 and 1896, and during 1897, but none of these caves were serious.

In 1899 some caves occurred above dike No. 1, and a contract was entered into for making and placing willow mattresses to prevent further caving at this place.

Work was started October 5, 1901, and two mattresses were built and sunk by January 29, 1902, one being 400 by 600 feet, and the other 400 by 450 feet, and loaded with 6,000 and 4,153 tons of rock, respectively.

The total cost of these mattresses under the contract was $52,794.93, making a total, with $1,253, for expenses in the office of the Chief of Engineers of $259,769.22 expended for bank protection to date.

In December, 1903, a cave developed above the mattresses placed in 1901-1902, the cave being about 1,000 feet long, and in 1905 two caves developed about three-fourths of a mile above the mattress work.

The tendency of this caving is to cause a flanking movement on the protection work which, unless prevented by extending the work up and down stream, may eventually destroy the present protection work and threaten the lock itself.

In November, 1908, a cave occurred between dikes Nos. 3 and 4 at the river approach, carrying down the temporary levee in front

of the lock and also the tower of the cableway belonging to the contractor for the river approach.

At this time the lock was not in condition to withstand water pressure, but, as it was during the low water season, no damage resulted from the destruction of the temporary levee, and the lock was placed in condition to withstand pressure when the spring rise reached a height sufficient to flood the river approach.

An eddy exists at the entrance to the river approach, where the last cave took place, and further caving is likely to occur there unless measures are taken for its protection, which can now be taken up as the river approach is practically completed.

THE PLAQUEMINE LOCK.

The essential difference between the lock connecting the Mississippi and Bayou Plaquemine and those in use in a canalized river is that in the latter there is, at the lowest water, a difference of level, dependent on the height of the dam, between the lower and upper pools, while at Plaquemine at low water the level of the bayou or lower pool is practically the same as that in the Mississippi or upper pool, and at such times the gates of the lock could all be opened and boats pass freely from the bayou into the river, or the reverse.

Such a condition requires a lock with the sills of both the upper and lower gates at the same elevation, and consequently both the upper and lower gates of the same dimensions. During high water the difference of level of the two pools at the Plaquemine Lock is due to the greater rise of the main river as compared to the bayou.

At the present time the records show the greatest rise of the Mississippi at Plaquemine to be about 37 feet, while the rise of the bayou, due to back water from the Atchafalaya, is about 9 feet.

The plans prepared by Major Stickney, in his report on a waterway from Plaquemine to the head of the Atchafalaya, included a double lock of brick at the head of the bayou, consisting of two chambers, each 300 feet long and 75 feet wide, with gates of the rolling type similar to those planned for use in the lock at Davis Island on the Ohio River.

The walls of the inner or bayou chamber were 25 feet high, while those of the outer or river chamber were 40 feet high, as it was intended that the river chamber should be used for all stages, from

low water to 15 feet in the Mississippi, and both chambers for stages exceeding 15 feet.

In the examination of Bayou Plaquemine by Major Heuer, borings were made to determine the character of the materials composing the foundation of the proposed lock near the head of the bayou. These borings extended to a depth of 18 feet below the low water places, and it was found that the foundation consisted, generally speaking, of layers of sand and clay, or a mixture of the two; the material near the low water level being quite stiff, and it

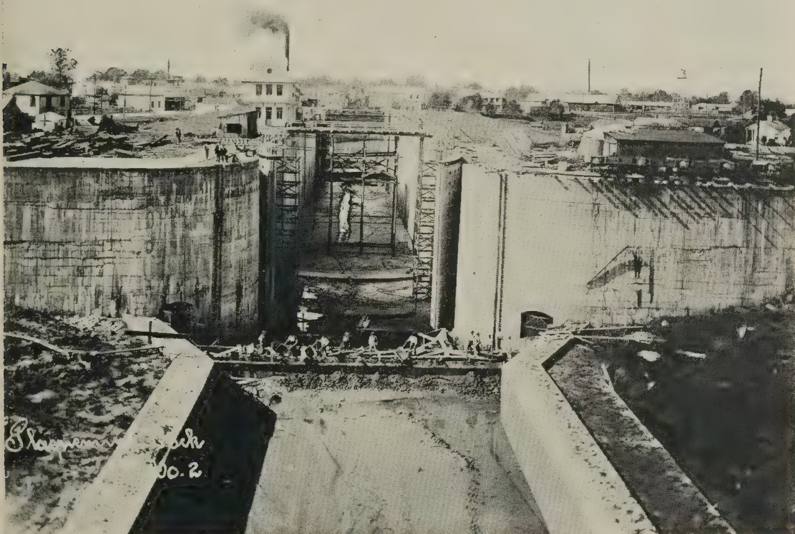


Fig. 2. River entrance to Plaquemine Lock.

was estimated that a foundation of piles on 3-foot centers might be safely subjected to load of 4,000 pounds per square foot. Lack of funds prevented actual work being started on the lock for some time after Congress began to make appropriations for Bayou Plaquemine.

With a part of the funds appropriated in 1890, however, a plan was prepared by the officer in charge for commencing work on the lock and submitted in May, 1891, recommending that the lock be located in the bed of the bayou in front of the public square at Plaquemine. It was proposed, if the site was adopted as recom-

mended, to cut through to the river so as to form the river approach, as the bayou at this point turned sharply to the north and joined the Mississippi about 1,000 feet farther upstream, the dike closing the bayou being located in the reach above the proposed lock site.

The plan of the district officer for the lock and its location were referred to a board of engineer officers for consideration and report, with instructions to furnish drawings of the lock adopted. The board recommended the adoption of the site proposed by the district officer and a lock 55 feet wide, with a clear length of 265 feet, a depth over the sills of 9 feet at low water, and with walls 42 feet high, the height of the walls being based on the extreme fluctuation of the Mississippi River as shown by the records at that time, namely 30.5 feet.

It was also recommended that the gates be of iron and double, in order to withstand the head during high water, and that they should be all alike, their operation being by hand by means of spars. The recommendation of the board further called for miter sills of cut granite bolted to the concrete floor of the lock with suitable timber cushion pieces, for cylindrical filling valves of the Muskingum type, for recesses for timber cofferdams in the fore and tail bays, and in the walls for ladders and two or more rows of mooring hooks.

The construction of the lock, recommended by the board, was to surround the site with sheet piling, excavate to the proper depth and then compact the earth in the foundation by driving 25 to 30 foot piling on 3-foot centers. The heads of the piles were then to be covered by a mass of concrete, which would extend in the wall to the low water surface, the floor and walls of the lock being monolithic to that height.

Above the low water surface a structure of brick in cement mortar with relieving arches to reduce weight was recommended, as some doubt was expressed about mass concrete retaining its integrity when not submerged.

The board estimated the cost of the proposed lock, including the purchase of the necessary land for its site, at $700,000.00. No work could well be started at the time the board submitted its recommendations, as sufficient funds were not available, but expropriation proceedings were begun to secure the necessary land for the lock site and the land was secured by June, 1894, at a cost of about $35,000.00.

With the funds appropriated in 1894, about $114,000.00 became available for lock construction, and plans and specifications were prepared for the work of constructing the cofferdam and for the excavation and piling required for the lock foundations. These

Fig. 3. A close view of crack in floor of lock and gates.

plans were based on the recommendations of the board and required the excavation of about 92,000 cubic yards of material, the surrounding of the site with triple-lap Wakefield sheet piling 35 feet long, and the driving of 8,800 foundation piles 50 feet long on 3-foot centers.

Contract for this work was entered into with the lowest bidder, E. A. Burriss, of New Orleans, in June, 1895, his bid having been $93,250.00.

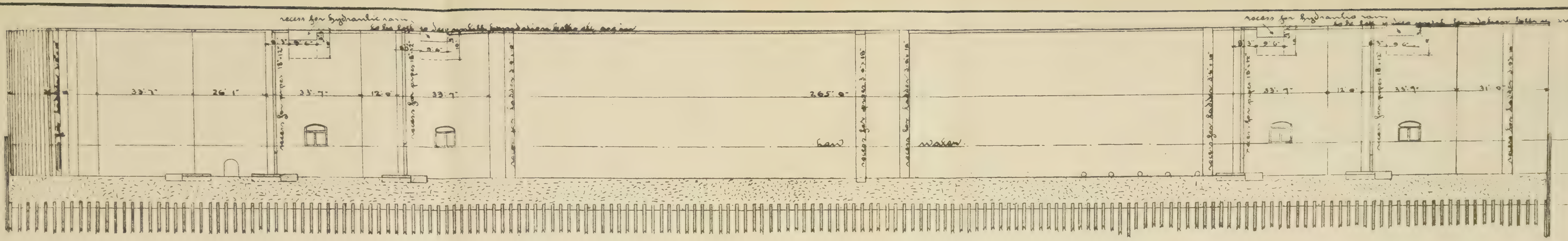
Work began in September, 1895, and by June of the next year some 83,000 cubic yards had been excavated and some 600 piles driven. In June a severe storm washed a large quantity of the excavated material back into the pit, and in August a dangerous movement of the earth occurred at the river end of the pit, which drove a large part of the sheet piling out of line so badly that it had to be redriven, and threatened the safety of some buildings in the vicinity of the lock site. At the town market-house a crack 8 inches wide developed alongside the building, and the pavement through the center of the structure opened at the joints about 1 inch.

The contractor was required to stop work and redrive and re-brace the sheet piling, as soon as the movement of the earth was noticed, and by amendment of his contract was given the extra work of removal of the earth, about 60,000 cubic yards, which the slip made necessary. The time limit of the contract was extended several times and the contract was finally completed in December, 1897.

Another slip of earth occurred in December, 1897, which drove in a large portion of the sheet piling surrounding the foundation, the piling not being broken off, but overturned as though offering no resistance to the movement of the earth whatever.

The height reached by the river at Plaquemine during the high water season of 1897 necessitated a complete revision of the lock plans, and, accordingly, new plans were prepared by the district officer in charge of the improvement for a lock of the same dimensions as recommended by the board, but with a depth over the sills of 10 feet at low water, and with walls 52 feet high, the construction being of concrete throughout.

The revised plans having been approved, contract was entered into with Stewart & Co. of St. Louis, Mo., in June, 1898, for the lock construction, the contract price being $501,787.60, and work began in August, 1898, in the removal of the earth from the pit, due to the slip in December of the previous year, and restoring the alignment of the sheet piling. Concrete laying began in June, 1899, and by November, 1900, the floor and walls had been completed, with the exception of the recesses in the latter for the gate-operating machinery.



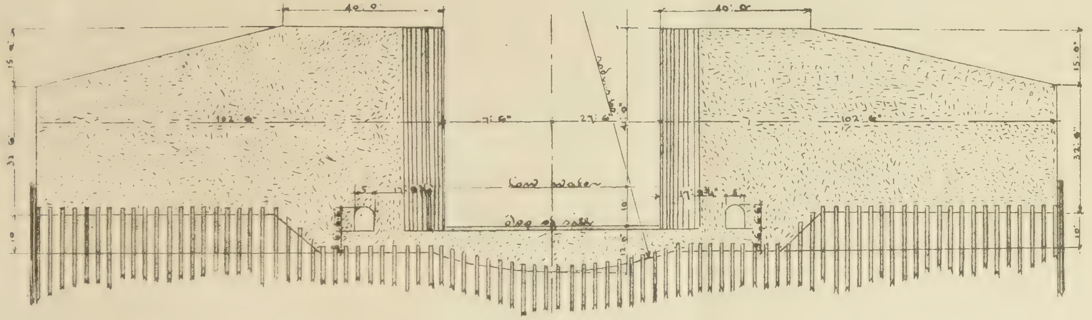
Improving Bayou Plaquemine

SECTIONS OF LOCK

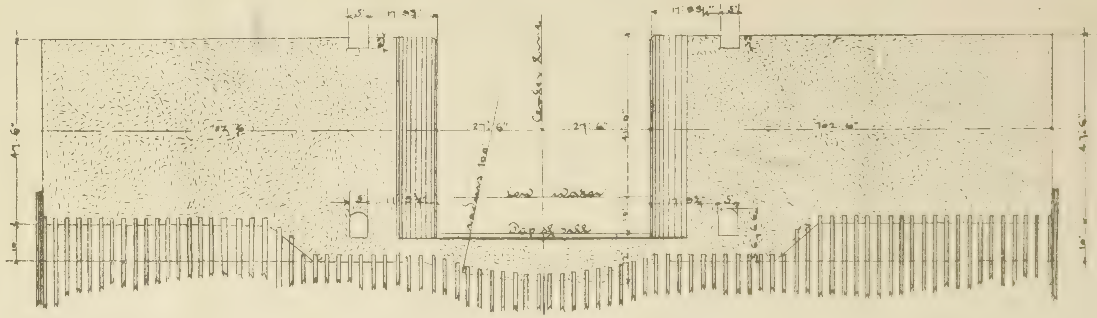
Longitudinal Section:
On line A.A.

Made under direction of Major James B. Quinn, Corps of Eng'rs. U.S.A.

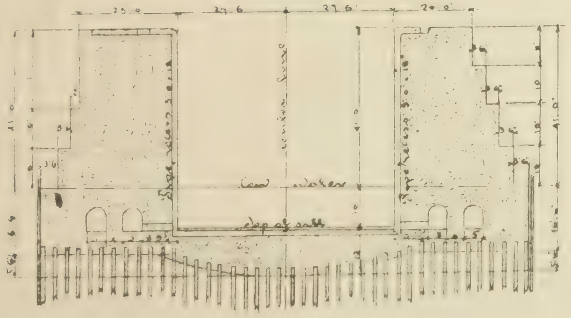
P.H. Thomson, Ass't Eng'r.



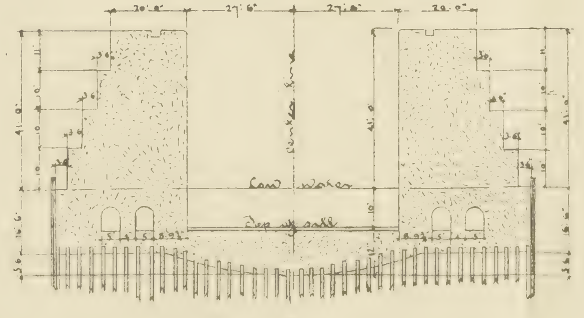
Section thro. Wing
On line B.B.



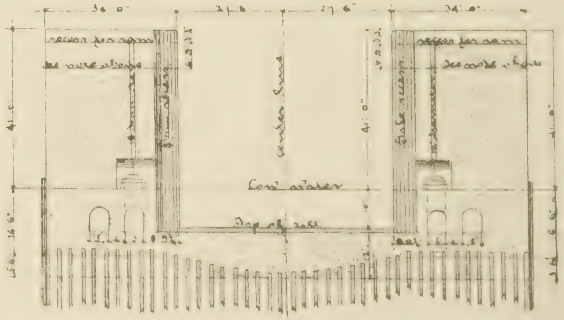
Section thro Wing
On line C.C.



Section
On line E.E.



Section
On line E.E.



Section
On line D.D.

Work was stopped on November 15, 1900, due to the settlement of the structure and spreading of the lock walls, which had caused a crack to develop in August of that year, the crack being in the floor of the lock and along the axis of the structure, though branching from the axis in places. Levels, taken at the time the crack was discovered, showed that considerable settlement in the whole structure had taken place before the crack developed, and also that the spreading of the walls had caused the sills to rise considerably in the center.

By the end of June, 1901, the total settlement of the structure along the axis of the lock was 0.26 foot, and the sills were 0.18 foot higher at the center than at the walls.

Two 100-ton test loads of pig iron, each of which represented approximately the weight of one leaf of the main gates, were placed on two of the pivot stones of the sills and allowed to remain for some time, but no additional settlement was apparent due to this loading.

Work on the lock remained practically at a standstill until August, 1904, but during 1902 a levee was constructed, under contract, connecting the northeast corner of the lock with the Mississippi River levee in front of the lock, and under the same contract a quantity of back fill was placed behind the lock walls.

Two small plots of ground, comprising 1.56 acres were purchased in 1901-1902, as they were required for the construction of the proposed river approach. Certain modifications of the lock gates included in the contract with Stewart & Co. were desired, but no agreement could be reached with the contractor and his contract was terminated in June, 1903, by the payment to the contractor of his retained percentage, which, with payments previously made him, aggregated the sum of $331,958.22 paid under his contract. During the existence of the contract with Stewart & Co. the question arose as to whether the cement supplied by the contractor was an American Portland, as required by the specifications. The question was referred to a board of engineer officers for decision, and after an exhaustive investigation, the result of which is published in War Department Document No. 119, it was decided that the contractor's cement, a product of the Illinois Steel Company, was not a true American Portland and should be rejected.

The termination of Stewart & Co.'s contract left the lock practically completed, with the exception of the gates, the bayou and river approaches and back-fill, and it was then proposed to con-

struct the gates under one contract and the approaches and backfill by others.

Contract for the lock gates was entered into in February, 1904, with the Penn Bridge Co. for $142,028.34, and the work of cleaning up the lock in preparation for the work and for the work of installing power house and operating machinery was started in August, 1904. Contract for the power house and operating machinery had been let in December, 1899, with the Otis Elevator Company, for $114,000.00, pursuant to the revised plans of 1897, which provided suitable machinery for operation of the gates. The work on the contract could not be started at that time and when arrangements were finally made for the construction of the gates the contractor was notified to proceed with this work and at once made preparations to do so.

The lock gates were completed in May, 1906, after considerable delay, due to the yellow fever epidemic of 1905, and the power house and operating machinery were completed in August of the same year.

The gates as constructed are ten in number, consisting of four pairs of main gates, and a pair of guard gates opening toward the bayou at the bayou end of the lock. The gates are of steel and of the mitering type and so curved that the center line is an arc of a circle with a radius of 48 feet 9 inches. They are formed of built-up horizontal girders separated by vertical diaphragms and sheathed on both sides, the sheathing on the inner or concave side forming the water-tight skin. They measure in thickness from out to out of sheathing plates about 2 feet 6 inches, and each leaf measures on the chord from center to center of contact plates 32 feet 1¾ inches. The four pairs of main gates are 50 feet 5 inches in height and are exactly alike, while the guard gates have a height of 20 feet 6 inches. The contact plates at the toes and heels of the gates are 2 by 8 inches, those at the toes abutting, when the gates are closed, while those at the heels abut against flanged hollow quoin plates secured to the masonry, and having 8-inch bearing surfaces. The main gates have wooden gangways with pipe railings over their tops and two pairs are also fitted with sluice valves, which may be operated by hand from the gangways. The main gates are operated mechanically while the guard gates, being habitually left open, are moved by hand when it is desired to close them.

The operating machinery for the gates and filling and emptying

valves consist of hydraulic engines furnished with water under pressure from a high pressure accumulator in the power house, the accumulator being supplied by high pressure steam pumps. The eight hydraulic engines for the operation of the main gates open and close the gates through their swing by one stroke of the piston.

The power house is located on the south wall of the lock, special provision for its location having been made in the plans for the south wall. It is of enameled brick and contains in addition to the power plant the necessary rooms for offices, etc.

At the time of the construction of the lock gates the walls had spread at their tops so much that it was necessary to set out the hollow quoin plates from the main walls so as to make the gates miter properly. It had been intended after the construction of the gates to first test them, and then complete the river approach and open the lock to navigation, and in order to prepare for the test and at the same time have as little of the river approach work as possible to be completed after the test of the gates two contracts were entered into in 1905, one of which provided for constructing a levee from the southeast corner of the lock to the Mississippi River levee in front of the lock, and for making back-fill behind the lock walls for about 100 feet from the river wing walls. This levee would, with the one constructed in 1902 on the north side of the river approach, form with the lock itself a basin surrounding the river approach, which could be filled with water during high river by cutting the Mississippi River levee between the two spur levees to the lock.

The other contract was for constructing the bayou approach and such portion of the river approach as could be built at that time.

The bayou approach consists of two concrete retaining walls 25 feet high and spaced 106 feet apart at the lock, and 135 feet at the lower end, following the curve of the bayou for a distance of about 300 feet from the bayou end of the lock, and a concrete apron between the walls to prevent scour due to the current from the lock.

The river approach consists of two concrete walls 60 feet apart extending from the river end of the lock toward the river, the earth floor of the approach between the walls being compacted by piling.

The spreading of the walls continued, and seems to have been aggravated by the back-fill placed under these contracts, and the

gates began to separate at their tops, which made it necessary to omit the proposed test of the gates, and, accordingly, another contract was made for completing the river approach work and the back-fill to the lock.

This contract was entered into in 1906 and was secured by the contractor for the bayou and river approach work let in 1905 and made three separate contracts in operation at the same time for work in the immediate vicinity of the lock. The other contracts should, according to their terms, have been completed when that of 1906 was let, but the contractors made very poor progress on this work, and that for the levee and back-fill was completed in January, 1907, and for the bayou and portion of the river approach work in January, 1908.

Settlement and spreading of the lock walls continued and resulted in another crack appearing in the floor near the north wall, and several appearing in the wall of the lock itself.

A board of engineer officers was convened in May, 1907, to investigate conditions and submitted their recommendations that the back-fill to the lock be completed as soon as possible, and that during the making of the back-fill the tops of the walls should be tied together to prevent or minimize further spreading, if possible.

After the back-fill was completed it was recommended that the lower lock gates be calked, the lock filled with water and left for about a month, after which the lock was to be emptied and the gates readjusted.

Arrangements were made for carrying out the recommendations of the board, the tops of the lock walls were tied together by means of seventy 2-inch steel rods with turnbuckles, and by special arrangements with the contractor for the river approach work some 24,000 cubic yards of earth were secured from an old levee near the lock and the back-fill completed in April, 1908, the back-filling extending to the tops of the walls.

Water was let into the lock in May, first by pumping into the partly excavated river approach by an hydraulic dredge, and, later, by siphons and cuts in the levee in front of the lock, and a depth of about 43 feet was held for about a month.

In July, 1908, the lock was unwatered and the work of cleaning out the débris and sediment, which had accumulated, was begun, in order to commence the work of readjustment of the lock gates, which at the time the work of carrying out the recommendation of

the board was begun, had separated, due to spreading of the walls, about 1 foot at the tops and 2 inches at the bottoms of the toe plates.

By another agreement with the contractor for the river approach work, it was decided to omit the further construction of the walls and piling of the river approach and he was permitted, when the lock gates had been readjusted, to dredge the balance of the earth in his contract, yet unexcavated, and dump it in the Mississippi.

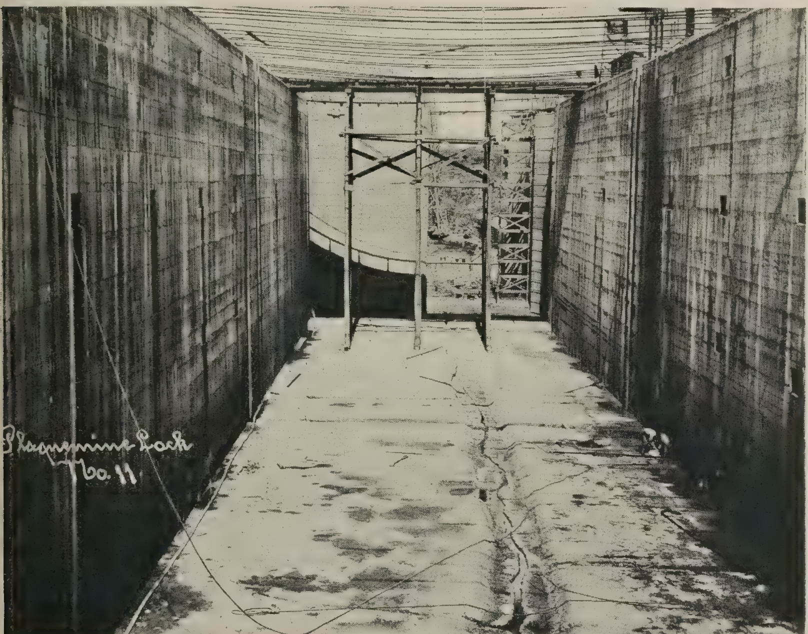


Fig. 4. Showing crack in floor due to unequal settlements, and 2-inch rods across top of lock to draw walls together.

The work of readjusting the lock gates was begun as soon as the lock was cleaned, and pushed as rapidly as possible, becoming emergency work in November, when the cave in front of the lock carried down the levee and exposed the work to flooding during the next high water.

The steel tie rods were removed before the work of adjusting the gates began, as it was desired to permit the structure to develop any tendency to further spreading before the gates were finally adjusted. Cracks, which had already developed in the structure, were cleaned and filled with cement grout and concrete.

The gates were lifted from their pintles by hydraulic jacks and the hollow quoin plates set out so as to be vertical, the space behind them being filled in the lower part with babbit, and in the upper part with rich cement concrete, arrangements being made in the anchor bolts of the hollow quoin plates for further adjustment should it ever become necessary.

The old gate anchorages had outlived their usefulness in so far as adjustment was concerned, and new anchorages were purchased and set in place.

When the gates were first erected and swung, in 1906, it was found that the clearance under the gates, which had been planned to be 6 inches, had diminished due to the rising of the floor in the center of the lock, to about 2 inches, and it had been necessary to cut away considerable concrete under the swing of the gates. The clearance then obtained had been considerably diminished, due to the further spreading of the walls and consequent rising of the floor since that date, and it was necessary to cut away considerably more concrete under the swing of the gates.

After the gates were reswung from their pintles it left the toes about 2 inches apart, and this was taken up by placing steel channels over the toe plates and filling behind with babbit in six of the gates and with oak filler pieces in the other two and the guard gates.

The operating engines were reset so as to operate the gates, and by the early part of March, 1909, the readjustment was completed and the lock filled with water by first cutting the bayou coffering dike, and later, when the river rose so that it was about to flow into the river approach, by cutting channels so as to permit it to fill the approach without washing in an excess of material.

Work of dredging out the river approach was begun in April, and has been completed since, but with the subsequent rise of the river boats were enabled to use the lock before work was finished.

Since the last adjustment of the gates no spreading of the walls, of any consequence, has taken place, and the settlement of the structure has been very slight and it is hoped that no further trouble from these causes will arise.

The walls, up to the time of the final adjustment of the gates, had spread at their tops from 1.6 to 2.6 feet and at the bottom from 0.2 to 0.4 of a foot.

The admission of water into the lock, by relieving the foundation

of a certain amount of pressure, was undoubtedly instrumental in stopping or reducing the settlement, and the back-fill has a tendency to prevent further spreading of the walls.

The gates have operated nicely since the final adjustment, but during periods of high water are interfered with considerably by drift in the river approach and deposits of sediment which must be removed on the recession of each high water.

The cost of the lock, including land for site, power house and operating machinery, approaches, etc., is approximately \$1,150,000 and the total expenditure on the whole Plaquemine improvement to the end of June, 1909, amounted to \$1,704,224.45.

Thermit Welding in Galveston District

BY

Mr. S. E. Lawrence
Junior Mechanical Engineer

The continuous operation of dredging plants, day and night, and the extreme usage that some of the machinery is subjected to is conducive of many break-downs, which are very expensive both in repair cost and more especially in delays and crew costs while parts are being repaired.

Such accidents are provided against as far as they can be ordinarily foreseen by keeping on hand such duplicate parts as may be expected to give way under usual conditions, and each plant in this district is equipped with a small machine and blacksmith shop to handle emergency repairs.

Such precautions do not, however, provide for the larger breaks and the extraordinary accidents, and it is in this particular field that the Thermit welding process has been of service in several instances.

The first time this method was employed by this office was in the repair of the low-pressure connecting rod of the 550-horsepower triple-expansion engine directly connected to the dredging pump on the 20-inch United States pipe line dredge *Col. A. M. Miller*.

This fracture was an example of the sudden development of an invisible flaw made in the original forging of the part. This engine had been in constant operation for several years and no precaution had been thought necessary to keep an extra rod on hand.

As it would delay the dredge for an indefinite time to secure a new part from the factory, and also several days at least to machine one in a local shop, it was determined to try and reweld the broken fork. Only fair results were obtained, and it was not thought advisable to use the rod. Then it was that the Thermit process was resorted to. About 1 inch of metal was removed and

the parts fastened to a bed plate in as perfect alignment as possible.

The weld came out of the sand in good condition and, after removing gate and riser, the part was placed in the engine without further machining and was so used with best of results.

The second instance in which this method of repair was made

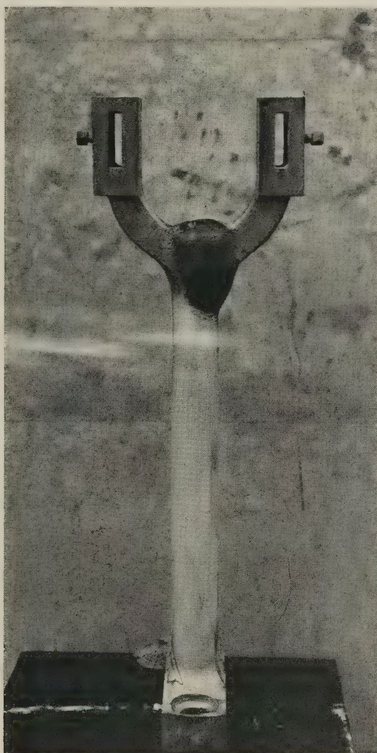


Fig. 1. Connecting rod of Dredge *Miller* (main engine), showing weld of fork.

use of was in the repair of one of the propeller shafts of the seagoing hopper dredge *Galveston*.

This was hardly a weld in the common acceptance of the term, as there was no uniting of severed parts, but it served as an illustration of the varied uses that this method can be put to.

While at work at the mouth of Galveston Harbor a wire cable became entangled about one of the shafts in such a way as to wear a groove in it about 1¼ inches deep and 4 inches wide. This weak-

ening of the 10-inch shaft necessitated its removal, a spare part being substituted.

The local shops did not afford a lathe of sufficient size to handle the removed shaft, and other methods of repair other than the conventional ones had to be resorted to.

Two processes suggested themselves, the Oxy-Acetylene process of antogenous welding and the Goldschmidt Thermit process, and representatives of both methods were asked to submit proposals for the filling of the groove cut by the entangled cable.

To facilitate matters the shaft was raised on concrete piers, as shown in illustration, the top of the piers forming a socket in which

Fig. 2. End of 10-inch propeller shaft of Dredge *Galveston* after finishing showing riser removed. (Arrow indicates position of fill.)

to turn the shaft more easily. This turning was accomplished by means of a tackle and rail.

Upon receipt of the proposals, the Oxy-Acetylene bid proved the lower and a contract was awarded for the filling in of the cut with new metal by their method.

Their apparatus was put in place and the shaft preheated with charcoal and covered with asbestos to retain the heat as much as possible.

The work was done under extreme physical conditions, the great heat requiring frequent relays and changing of men, it being almost impossible for a man to stand close enough to the cut to operate the burner for very long at a time.

When the circle was at last complete and inspected, the ring of

metal melted into the cut was found to be separated from the metal of the shaft in places, the bond was insufficient to strengthen the weakened place, and the material so placed was easily removed. The failure was due mainly to the large radiation and conducting of the heat from the particular part where a bond of metal was desired, and to the severe physical strain. Smaller parts have been successfully handled here.

The Thermit exponents were then given a chance to fill in the worn place. The groove was carefully filled with wax and an adjustible flask placed around the worn part and the mold made. Air was supplied from a derrick car, the derrick also being utilized

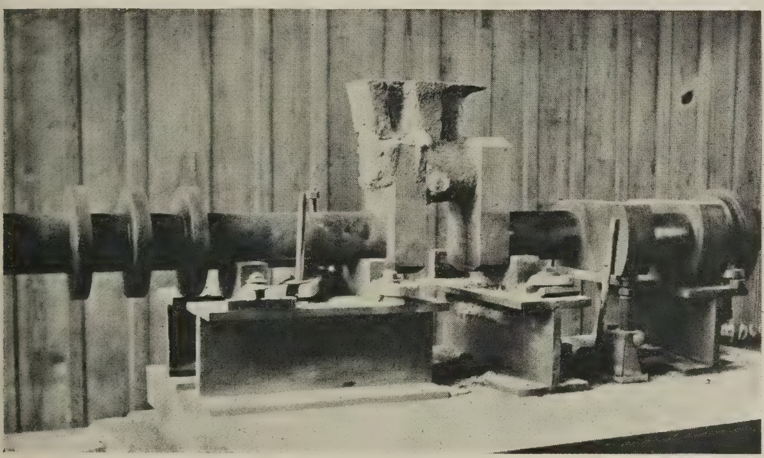


Fig. 3. Flasks removed, showing part of clay mould and riser.

to suspend the crucible containing the charge. The preheating was accomplished by gasoline blow torches and the wax was carefully removed and the mold cleaned by compressed air.

The reaction was perfect, and the result was a complete welding of the new metal into the groove. Precautions had been taken to provide a large riser and gate to cover the shrinkage and to insure solid metal in the worn part, as the top metal may be in some cases more or less porous.

When the mold was removed, very satisfactory results were found. The removal of the extremely tough surplus metal was the most difficult part of the work. The gate and riser were finally removed, and outside of a few marks the weld was unnoticeable. Some concern was felt about the chances of warping shaft when such an intense heat was suddenly applied in a comparatively short

length, but such fears were dispelled when no appreciable change in the alignment was evident.

No machine finishing was necessary in this case, as the portion of the shaft affected did not come upon any bearing surface. Herein consisted the efficiency of this particular method, as there was no machine of sufficient size to handle this length of shafting available in this immediate locality, which would have barred the ordinary schemes of repair.

The third incident of interest in this district was the repair of the crank shaft of the main engine of the United States Engineers pipe line suction dredge *Captain C. W. Howell.*

The crank shaft of this 12 by 22 by 14 inch compound engine directly connected to a 12-inch dredging pump was 5½ inches in diameter. A crack was discovered in the low-pressure crank-pin, and a careful investigation showed that instead of the characteristic fracture the crack ran into the web and back again around the pin. This precluded all the ordinary schemes of replacing the broken pin by shrinking in a new one, as the web did not have sufficient remaining metal to make such a repair safe.

The dredge was operated until arrangements could be made for a Thermit weld, and on the repair day the plant was shut down and shaft removed to a shop. It was found to be entirely broken off. Part of the pin was machined off, leaving about an inch of space between the web and pin when the parts were aligned and fastened to a bed plate.

The mold was prepared in the usual way, and the ends to be united heated to as high a temperature as was safe.

There was no hitch in the pouring of the crucible and, after cooling over night, the mold was removed in sections to prevent uneven contraction.

When the gates were removed and the complete shaft placed in a lathe the nice allowances for shrinkage, etc., was evidenced by the fact that scarcely any refinishing was necessary on the thrust collars of the shaft. The pin was turned up and the surplus metal removed and web reshaped to original dimensions without changing the balance of the shaft.

A total of four days was lost by the plant because of this repair, most of the delay being due to the distance of the dredge from the shop.

This shaft continued in constant use without any evidence of the

repair whatever, and was in use when this dredge was lost a year after on the Texas coast.

An idea of the saving effected in this instance may be obtained from the fact that a new shaft ordered rushed at once from the factory at a cost of $465.00 was not ready for shipment ninety (90) days after receipt of order, as compared with a charge of $150.00 and a delay of four days.

The necessity of skilled handling and placing of parts to be welded can not be too forcibly emphasized, as the chances for ruining a part are greater from mishandling than from failure of the weld, and much unnecessary finishing work is prevented by careful attention to the mold conforming exactly to the desired shape of the part to be repaired.

The hard driven machinery of dredging plants, with their interdependent individual machines, offer an excellent field for economic employment of this method, and it will very likely play an important part in cutting down long delays brought about by extraordinary accidents.

The Three-Point Problem and Hydrographic Surveys

BY

Mr. JAMES P. ALLEN*
Assistant Engineer

To the hydrographer, especially in harbor engineering, the sextant is the most convenient instrument for general uses. It serves him for locating soundings, wrecks, points on jetties and on outlying islands, lights, etc. To locate a point by measuring angles at the point itself, he must have at least three well established points, which may be on shore or on jetties, light-houses, etc. By means of these, the position of the observer may be determined within certain rather broad limitations, and with a precision greater or less as circumstances may require or conditions permit.

The method of location is simply this: At the point to be located the observer measures the angles between the known objects. The point to be located can then be platted by a three-arm protractor (station pointer) or by some other graphical method. Its position may also be calculated by what is known as "the three-point problem." A high degree of precision may be secured by the use of a transit, where this may be found practicable. The main object of this paper is to explain a method of solving this problem, which, to the writer seems simpler than those usually given in books on surveying. For example, the formula in Lee's "Tables and Formulæ," while apparently simple, requires great care in the proper application of the signs of some of the trigonometrical functions. For most practical men this is a difficult thing to do, and much time may be lost on account of confusion in the use of these signs.

SOLUTION OF THE THREE-POINT PROBLEM.

A, B, and C are the three known points.

P the point to be located.

a and b are the measured angles.

Draw the diameters BD and BE. The angle ADB$=a$, being

* Charleston, S. C., Engineer District.

inscribed in the same arc of the circle ABPD. Similarly, the angle BEC=b. The diameter BD= $\frac{AB}{\sin a}$ and BE= $\frac{BC}{\sin b}$. The angle DBE = ABC — (90°—a+90°—b) =ABC + a + b —180°. Connect the points D and P, and also E and P. As the angles BPD and BPE are inscribed in semi-circles they are right-angles, and the line DPE is a continuous straight line perpendicular to BP at P. In the triangle DBE the two sides DB and DE and the included angle DBE have been determined, from which can be obtained the remaining angles BDP and BEP. From the known azimuths of BA and BC the azimuths of the diameters BD and BE are readily determined, and from these the azimuths of DE

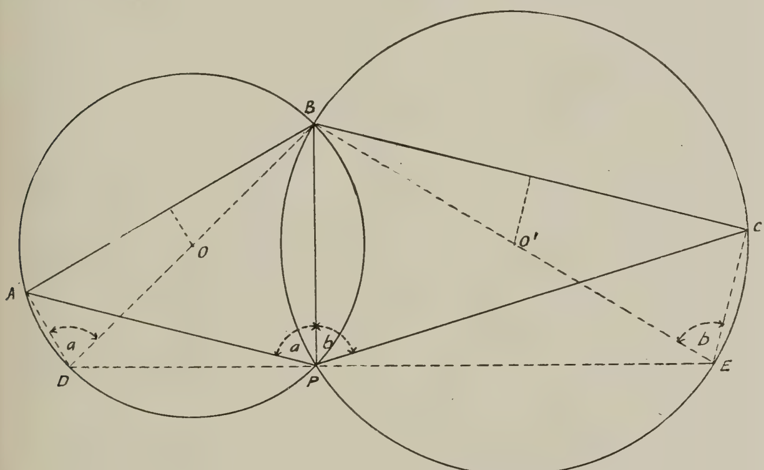


Fig. 1. Diagram for three-point problem.

and BP. The length of the line BP may be found from the expressions BD sin BDP, or BE sin BEP. Having now the direction and length of BP, the position of the point P is determined.

The accompanying form may be used for the calculation. Generally, the problem of location from three known points can be solved more quickly and with sufficient accuracy by graphical methods.

The following partially graphical method has been found convenient and fairly accurate: Erect perpendiculars at the middle points of AB and BC. Lay off on the perpendicular to AB a distance equal to ½ AB cot a. The point so found is the center of the circle ADPB. Similarly for the circle BPEC. With dividers

or beam compass, using radii BO and BO′, describe arcs intersecting at P the point desired.

It has been customary at Charleston Harbor, S. C., to locate soundings by one transit set on shore or on one of the jetties, and a cut-off sextant angle taken on the sounding-boat. This sextant angle determines a circle. The corresponding direction given by the transit crosses the arc at the point to be located. Diagrams on a large scale have been prepared, giving circles for various sextant objects at degree intervals, the centers being determined by the method given in the last paragraph. The minutes read by the sextant are interpolated proportionally between degree circumferences. Tracing cloth is laid over the diagram and sounding positions are platted on the cloth. This or a similar method was used at Fernandina, Fla., and a description of it is given in the technical details of the report of the Chief of Engineers for 1902 (page 2513, Report of Chief of Engineers). It has been used in the Charleston office since 1885.

In preparing these diagrams for a large scale map it will be found difficult to get the circles to agree closely with the coordinate lines on a sheet laid off for coordinates. If the positions of the sextant objects platted on the sheet are used in taking off the radii, the circles will not check closely with the coordinates. To obviate this difficulty, points on the circumferences can be calculated which will be in the field where the work is actually done. The line perpendicular to AB at its middle point forms the line of centers and is often convenient for this purpose. The calculations giving intersections of the several circumferences with this line can be made by the use of the formula that is here given for centers, viz, $\frac{1}{2}$ AB cot a. The 90° circumference passes through the center for 45°, the 88° circumference through the center for 44°, and so on.

It will also be found that scaling along the perpendicular can not be very accurately fitted to the coordinates, due to changes in the paper. It is sometimes best to coordinate the center and circumference of every tenth circle. The intermediate centers and circles may be interpolated by the use of a table of natural tangents. It appears that the preparation of one of these diagrams involves considerable work. This is true, but if made on good mounted drawing paper one can be used for a long time. There is one in the Charleston office, still serviceable, which is twelve years old. This method of surveying by sextant and transit with the accom-

panying plan for platting positions, as described, is not well suited to a general hydrographic survey. It is admirably suited for the sounding of a channel which is in process of formation, either by natural forces or by dredging, and in which it is desired to make frequent surveys over the same area. It is especially simple at Charleston Harbor for the reason that sextant objects can be ob-

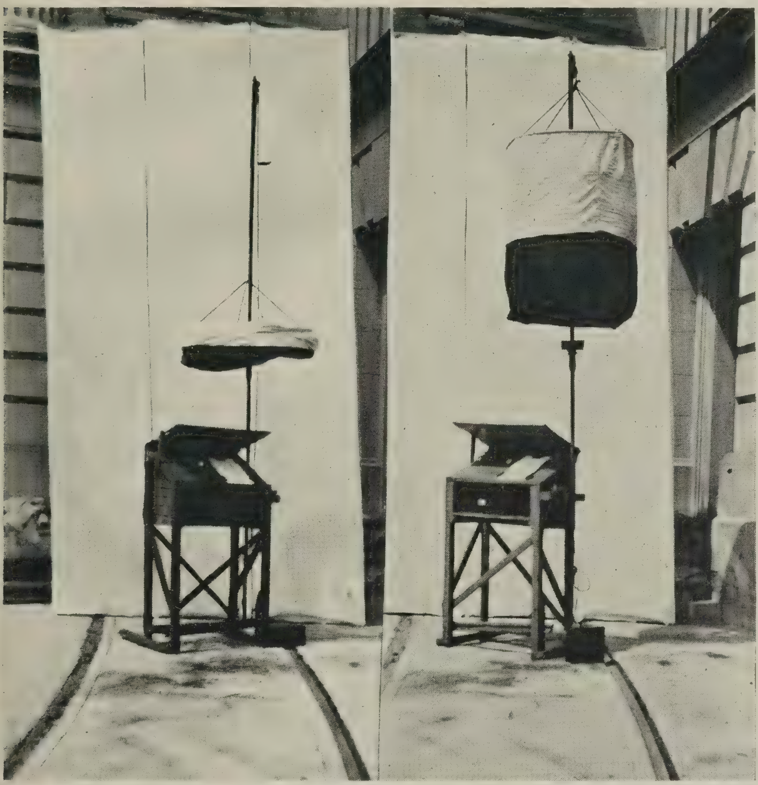


Fig. 2. These views show sounding target used at Charleston, S. C.; first, when closed and, second, when spread. The bright object in the desk is the sextant, which is placed there to leave the observer free for other work when not actually using the sextant. The note book, held open by strong paper clips, and the watch are also shown on the desk. The cover of the desk can be raised or lowered by means of the notched support on left. This protects the book from the sun and from the splash of the lead line.

tained by using permanent structures on Morris and Sullivans islands.

The surveys at the Charleston, S. C., entrance have been made by a party consisting of four or five men—a transit-man on Fort

Sumter or on one of the jetties, a sextant-observer, who also keeps all of the notes taken on the boat, reads sextant angles and gives the signal; a sounder, and a steersman. If the sounding boat is large enough to require an engine-man, the party consists of five men.

The signal is made of a series of iron or wire-rope hoops. The top, middle, and bottom hoops have two wooden pieces crossing each other at right-angles and firmly connected at the center, and to the outer ends of these the hoops are attached. A hole is made in the center of these arms or spreaders, large enough for 1-inch pipe to pass through easily. White and black cloth in strips is attached to the outer circumference of the hoops, making a cylindrical hollow target about 3 feet in diameter and 3½ feet in length, which can be spread or closed at will. When in use this target slides up and down on a piece of galvanized iron pipe about 1¼ inches in diameter, called the mast. This mast is attached to a desk at which the observer stands. This desk is heavily built, and is provided with a tripping arrangement by which the signal can be held up until released by the foot of the observer. The target is hauled up by the sextant observer by means of a line attached at its top. This line passes through a pulley at the top of the mast, thence through the target to the desk. The desk is strongly lashed to the deck of the sounding boat. When he wishes to make a signal for location the observer raises the signal, and in the act of raising he also spreads it. At the instant of observation he trips the signal with his foot. It falls and closes, so that to the observer on shore it completely disappears from view.

The positions of the sextant objects are such that the azimuth lines given by the transit from the station at Fort Sumter cut the circumferences at large angles.

The speed of the boat is regulated, so far as practicable, with reference to the proposed scale of the map and the object of the soundings. It is difficult, usually, to run naphtha boats slowly enough to secure sufficient soundings, when it is necessary to have them very close. On actual surveys made recently the spacing varies from about 50 feet to about 175 feet between soundings. The speed of the boat varies from about 5 feet per second to about 7 feet per second.

The lead-line is made from Irish hemp. This is sold in hanks for the purpose. It is made about ¼ inch in diameter and is usually made up to 10 or 15 fathoms in length. It is well stretched

before marking, but it generally becomes necessary to re-mark very soon. As the line continues in use it becomes more stable. It is regularly tested at the beginning and end of each day's work and sometimes oftener. No satisfactory substitute for this material, in spite of its marked disadvantage, has ever been found, so far as is known. The sinker is of pear shape, about 3 inches high and 2½ inches diameter near the bottom. It weighs from 7 to 9 pounds. The use of the regular ships' lead was discontinued years ago, because there was good reason to believe that it will lie over in a strong current. The sounder may lift it from the bottom before it straightens up, thus reading the depth too great by several inches at times.

THREE-POINT PROBLEM.

Given or measured. $<a$, $<b$, AB, BC $<$ABC		$<$DBE $=$ $<$ABC $+a+b-180=$ B	
log AB	deduct.	log BC	deduct.
log sin a		log sin b	
log d (BD)		log d_1 (BE)	
d		d_1	
$d+d_1$	$d-d_1$	$\frac{1}{2}(180-B)=\frac{1}{2}(D+E)$	
$\tan\frac{1}{2}(D-E)=\dfrac{\tan\frac{1}{2}(D+E)(d-d_1)}{(d+d_1)}$ (*Form.*)		log tan $\frac{1}{2}$ (D+E)	
		log $(d-d_1)$	
$\frac{1}{2}$ (D+E)	deduct for less	co-log $(d+d_1)$	
$\frac{1}{2}$ (D—E)		log tan $\frac{1}{2}$ (D—E)	
$\frac{1}{2}$ (D+E)	add for greater	$\frac{1}{2}$ (D—E)	
$\frac{1}{2}$ (D—E)			

Compute azimuth of line DE, and from this azimuth of BP

		check	
log d	add	log d_1	add
log sin D		log sin E	
log BP		log BP	
log BP		log BP	
log cos Az		log sin Az	
log Lat		log Dep	
Lat BP		Dep BP	

Cost, Longevity, and Repairs of Barges, Tow-boats, and Other Pieces of Floating Plant Used in the United States Improvement of the Upper Mississippi River, 1881-1911

BY

MR. C. W. DURHAM
Principal Assistant Engineer

During this period of thirty years, this improvement has owned and employed 282 barges (scow), 12 barges (model), 90 quarter-boats, office-boats and store-boats, 3 steam drill-boats, 4 dipper dredges, 5 hydraulic dredges, 7 pile drivers, 23 dump boats, 3 snag-boats, 16 tow-boats of various sizes, and a very large number of small steam and gasoline launches, motor and ordinary skiffs, pontons, and other small pieces.

It will not be practicable within reasonable limits to follow the destinies of so many pieces, and therefore certain characteristic groups of various kinds are taken, from the experience of which conclusions may be drawn. Pieces built within the last few years are not considered. I would say that none of the pieces up to 1908 had any kind of wood preserver except, occasionally, Carbolineum Avenarius laid on with a brush, but during the past three years 80 barges, 4 dumps, 3 dredges, 33 pontons, and 3 quarter-boats have been built, of which most of the lumber in the hulls has been treated with creosote by the open tank or dipping process. Sufficient time has not elapsed to show the value of this treatment.

In 1911, we are treating lumber in barge construction by a pressure process at Sandstone, Minn.

SCOW BARGES.

A small scale drawing of a barge 100 by 20 by 4½ feet is shown, the same being the standard applicable to all 100-foot barges hereafter mentioned. This district also uses a standard barge, 110 by 24 by 5 feet, of practically the same construction as the 100-foot barge, and for the purposes of this article the same design applies

to all barges and to quarter-boat and drill-boat hulls, except the model barges (12) and those bottom planked fore and aft (6).

The barges used in the earliest years of this improvement for carrying rock and brush, were mostly of smaller size than those at present employed, were built of white pine, and with caulking and nominal repairs, gave good service for periods ranging from eight to eleven years.

Small Barges Used Early in the Improvement. All Built of White Pine, Untreated.

No.	Size.	Builder.	Where built.	Year.	Cost.	Longevity	Remarks.
	Feet.					Years.	
1-8	80x16x4	Wilson	Prescott	1881	$560	9	With one exception, and that due to accident, these barges gave good service for 9 years, and several were used for carrying brush a few years longer.
10-12	65x16x4	Eckhardt	Davenport	1881	720	11	Gave good service for 11 years and brush service for several more.
20-24, 36-39, 47	81x16x4	Hired labor, U. S.	Clinton and Davenport	1881	----	9	Do.
25-35	66x16x4	Eckhardt	Davenport	1881	548	8-10	Do.
65-72	80x16x4	Do	Do	1882	685	9	Do.
76-85	80x16x4	Diamond Jo	Dubuque	1882	660	8	Do.

GROUP I.

Built by Isherwood, Davenport, 1891; 100 by 20 by 4 feet, White Pine. Cost, $770 each.

No	Repairs.										
	1892	1893	1894	1895	1896	1897	1898	1899	1900	1901	Total
15	$48	$51	$170	$278	$58	$52	$21	$3	Bad	Condemned	$681
19	0	0	104	8	58	169	2	Bad	Do.	Do.	341
37	48	51	152	26	60	152	16	Do.	Do.	Do.	505
44	0	14	32	0	60	63	91	Do.	Do.	Do.	260
78	0	0	0	0	59	46	Bad.	Do.	Do.	Do.	105
96	0	0	32	0	59	0	0	Do.	Do.	Do.	91
114	48	23	220	0	60	149	0	3	Do.	Do.	503
117	0	29	185	0	58	131	49	0	Do.	Do.	452

With one exception (78) the good life of this group of barges was seven years. The large repairs on four barges were due to accidents, collisions, snags, etc.

GROUP II.

Built by Whitney, Rock Island, 1891; 100 by 20 by 4 feet, White Pine. Cost, $770 each.

No.	Repairs.										
	1892	1893	1894	1895	1896	1897	1898	1899	1900	1901	Total
1	$48	$51	$192	$29	$58	$167	$16	Bad.	Bad.	Condemned.	$561
2	0	0	78	0	0	56	0	0	Do.	Do.	134
18	0	23	60	0	0	40	62	0	Do.	Do.	185
39	48	32	144	0	60	80	11	0	Do.	Do.	375
43	17	33	0	60	31	0	Bad.	Bad.	Do.	Do.	141
82	92	0	29	0	0	145	0	0	Do.	Do.	266
115	48	25	74	5	58	149	0	0	Do.	Do.	359
116	0	0	3	0	60	37	91	Bad.	Do.	Do.	191

With one exception (43) the good life of this group was seven years.

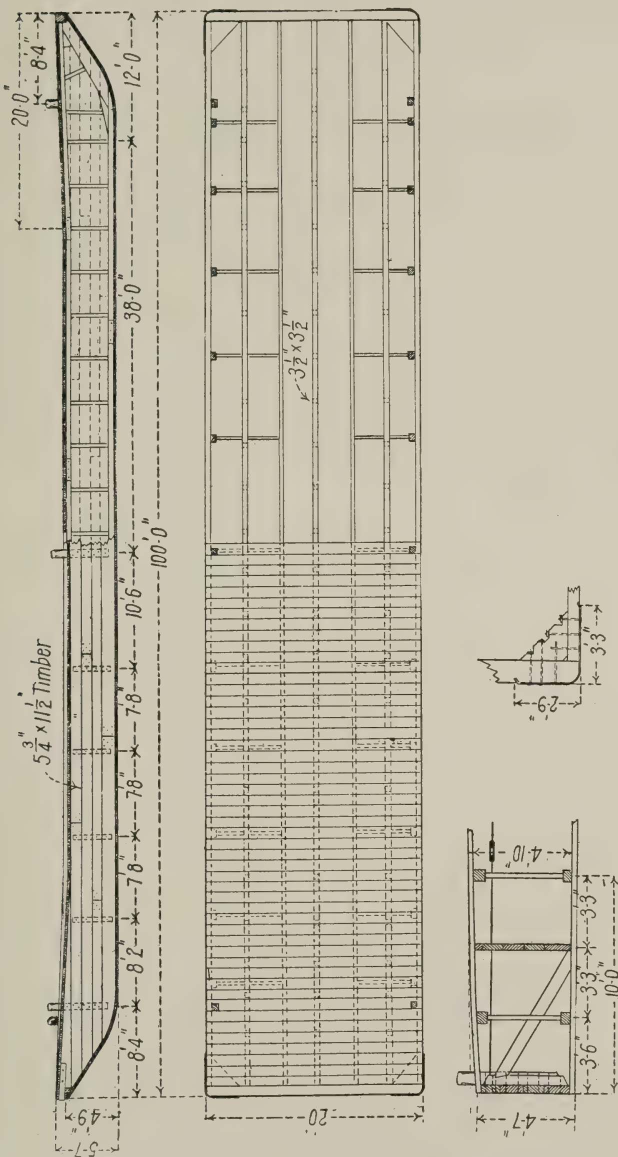


Fig. 1. Plans for standard 100-foot barge for rock and brush.

GROUP III.

Built by Kahlke, Rock Island, 1892; 100 by 20 by 4 feet, Douglas Fir. Cost, $806 each.

No.	Repairs.																	
	1893	1894	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	Total
143	-----	$7	-----	-----	-----	$2	$115	$66	$8	$406	-----	$160	$32	$44	$93	$93	*	$1,026
144	-----	50	-----	-----	42	34	139	215	-----	170	-----	99	30	74	19	144	*	1,016
145	-----	24	-----	-----	-----	37	124	200	8	144	-----	122	19	47	98	116	*	939
146	-----	100	-----	-----	16	155	220	115	-----	300	16	143	30	78	*	-----	-----	1,173
148	-----	108	-----	-----	8	90	160	155	-----	215	-----	151	90	126	*	-----	-----	1,103
149	-----	87	15	-----	6	82	40	64	-----	276	-----	8	50	**	*	-----	-----	628
150	-----	107	-----	-----	15	101	134	83	30	430	***	-----	-----	-----	-----	-----	-----	900
151	-----	78	28	-----	20	140	137	188	-----	279	41	189	174	10	*	-----	-----	1,284
152	-----	100	-----	-----	24	70	55	190	150	489	-----	50	170	25	*	-----	-----	1,323

Longevity fifteen and sixteen years, with deck repairs and partial rebuilding. *Condemned. **Bad. ***Wrecked.

GROUP IV.

Built by Batchelder, Stillwater, 1894; 100 by 20 by 4 feet, Douglas Fir. Cost, $800 each.

No.	Repairs.																	
	1894	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	Total
168	-----	-----	-----	-----	-----	$38	$17	$8	$352	-----	$52	$32	$27	$93	$45	-----	$36	$700
169	-----	-----	-----	-----	$33	-----	91	8	326	-----	56	16	26	90	37	$12	35	730
170	-----	-----	-----	-----	27	28	-----	14	85	-----	378	25	26	93	205	-----	*	881
171	-----	-----	-----	-----	35	23	-----	18	109	-----	350	23	4	70	74	**	-----	706
172	-----	-----	-----	-----	-----	22	17	51	385	$10	53	55	25	-----	10	72	34	734
173	-----	-----	-----	-----	-----	33	30	-----	234	76	52	16	18	-----	12	57	36	564

Nos. 168, 169, 172 and 173 were partly rebuilt in 1902; 170 and 171 in 1904. All of these, except 171 and 173 are now (July, 1911) in fair condition. All of these went eight years with merely nominal repairs, such as calking. Repairs surprisingly small.

**Condemned *Fair.

GROUP V.

Built by Kahlke, Rock Island, 1894; 100 by 20 by 4 feet, Fir. Cost, $806 each.

No.	Repairs.																	
	1894	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	Total
174	-----	-----	$24	-----	$34	$10	$112	$188	$115	-----	$84	$56	$36	$107	$192	*	-----	958
176	-----	-----	-----	$1	3	5	108	30	89	$15	251	38	19	102	**	-----	-----	661
177	-----	-----	-----	-----	10	110	204	-----	75	-----	422	14	21	*	-----	-----	-----	856

Good life; twelve years, with nominal repairs and calking. *Condemned. **Bad.

GROUP VI.

Built by Whitney, Rock Island, 1895; 100 by 20 by 4 feet; Fir gunwhales, remainder White Pine. Cost, $790.

No.	Repairs.															
	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	Total
185	$18	------	------	$54	$193	$30	$85	$41	$50	$45	$1	*	------	------	------	$517
186	24	10	------	102	17	18	73	----	--------	47	63	326	$225	$445	164F	1,514
187	18	------	------	55	84	9	393	41	50	165	15	645	192	119	3F	1,789
188	------	14	37	93	62	------	381	----	--------	70	61	107	104	57	56F	1,042
189	------	4	91	100	125	24	309	41	50	155	35	625	161	*	------	1,720

F=Fair. A good life, but repairs in later years large and perhaps unjustifiable. Three of these still in use, 1911.

*Condemned.

GROUP VII

Built by United States, Fountain City, 1895-1896; 100 by 20 by 4½ feet; Douglas Fir. Cost, $768 each.

No.	Repairs.															
	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	Total.
192	------	$35	$41	$68	------	$15	$88	$174	$114	$55	$49	$395	$145	$220	$276F	1,675
193	------	------	272	69	------	------	169	26	100	205	50	314	68	145	259F	1,677
194	------	------	------	69	------	------	134	48	36	357	49	442	284	105	259F	1,783
196	------	------	------	56	------	------	233	26	242	5	49	150	59	*	------	820
197	------	------	87	28	27	------	207	77	170	55	85	*	------	------	------	736
198	------	------	------	24	------	------	181	45	51	259	107	188	39	*	------	894
199	------	------	------	66	------	------	106	46	243	128	50	*	------	------	------	639
200	------	------	42	64	------	15	159	26	38	5	**	*	------	------	------	349
201	------	------	------	25	24	------	56	21	169	102	49	400	216	*	------	1,062

A good life, but repairs toward the end too large. Three still in use, 1911. *Condemned. **Bad F=Fair.

GROUP VIII.

Built by Brown, Quincy, 1892-1893; 100 by 24 by 4½ feet; Douglas Fir. Cost, $1,600 each.

No.	Repairs.																
	1893	1894	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	Total
153	-----	$31	$50	$7	$10	$61	$162	$312	-----	$230	-----	$86	$54	$33	$97	**	$1,133
154	-----	33	3	4	8	124	23	165	$153	101	$78	400	165	-----	***	-----	1,257
156	-----	18	2	-----	92	*	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
157	-----	18	35	25	8	121	23	151	144	70	78	400	165	35	60	***	1,333
158	-----	70	2	3	49	156	18	150	147	78	78	397	290	9	7	***	1,454
159	-----	34	2	4	14	156	23	168	192	51	78	400	231	5	-----	***	1,358
160	-----	34	2	10	14	153	23	159	126	76	78	397	200	5	26	***	1,294
161	-----	18	8	-----	14	63	160	317	-----	182	-----	96	69	41	118	***	1,086
162	-----	17	159	-----	-----	74	144	380	-----	211	-----	104	73	57	124	***	1,343

*Wrecked. **Rebuilt. ***Condemned.

Several of these barges were partly rebuilt and all had new decks. No. 153 rebuilt and in use, 1911.

GROUP IX.

Built by United States at Keokuk, 1894; 110 by 24 by 5 feet; Douglas Fir. Cost, $1,400 each.

No.	Repairs.															
	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	Total
4	$20	$88	------	$2	$17	*$327	------	$74	-------	$173	$33	$35	$79	**	------	$848
7	------	------	------	33	72	169	$3	*200	-------	178	39	19	89	**	------	802
70	------	------	------	------	------	24	49	*337	41	250	-------	99	116	***	------	916
179	------	1	2	------	44	21	13	*362	41	250	90	100	97	47	****	1,068
180	------	1	2	22	5	16	42	*375	41	50	127	86	87	****	------	854
181	------	1	2	22	5	33	49	*325	41	250	98	104	59	10	***	999

*New deck and partly rebuilt. **Rebuilt. ***Bad. ****Condemned.

GROUP X

Built by United States at Le Claire (C.W. D.), 1885; 120 by 20 by 5 feet. White pine with oak bottom, planked fore and aft. Cost, $1300 each.

No.	Repairs.																			
	1885-1890	1891	1892	1893	1894	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	Total
97----	----------	$90	$159	$47	*$1,229	$172	----	----	$101	-----	$100	$29	$308	$80	$391	$105	$49	----	**	$2,860
98----	----------	132	16	10	------	*901	----	----	----	106	216	5	160	-----	225	27	18	95	**	1,911
99----	----------	64	----	64	*949	18	----	----	34	114	355	-----	206	9	----	31	18	115	**	1,977
100----	----------	83	----	222	54	----	----	$14	70	-----	-----	**	-----	-----	----	----	----	----	----	443
101----	----------	171	159	31	*1,082	*394	----	30	6	74	46	-----	217	82	365	165	49	----	**	2,871
102----	----------	139	159	31	*1,034	----	----	210	6	2	34	-----	218	79	384	178	49	----	**	2,523

*Rebuilt; life 21 years, once rebuilt. **Condemned.

These were the best barges used in this improvement. Cost of repairs and rebuilding large, but justifiable.

RECAPITULATION. SCOW BARGES.

Group	No. of barges	Dimensions.	Material.	Cost each.	Years in service.			Repairs.			Remarks.
					Max.	Min.	Average	Maximum.	Minimum.	Average.	
		Feet.									
I	8	100x20x4	White pine	$770	8	6	$7\frac{1}{4}$	$681	$91	$367	Light loads for the last year or two.
II	8	100x20x4	White pine	770	8	6	$7\frac{1}{2}$	561	134	277	Do.
III	8	100x20x4	Fir	806	16	13	$14\frac{1}{4}$	1,323	628	1,044	Wrecked barge omitted. Barges show great vitality. Principal repairs, new decks and calking.
IV	6	100x20x4	Fir	800	17	15	$16\frac{2}{3}$	881	564	719	Great longevity. Small repairs, all but two in fair condition, 1911.
V	3	100x20x4	Fir	806	14	12	13	958	693	825	Not so satisfactory as III and IV.
VI	5	100x20x4	Fir and pine	790	15	11	14	1,789	517	1,316	Very large and apparently unjustifiable repairs in the last four years. Three still in use and in fair condition.
VII	9	100x20x4¼	Fir	768	15	9	$12\frac{1}{2}$	1,783	349	1,071	Built U. S. Three in fair condition, 1911; large repairs.
VIII	8	110x24x4½	Fir	1,600	15	14	15	1,454	1,086	1,282	Wrecked barge omitted; long life with moderate repairs.
IX	6	110x24x5	Fir	1,400	14	13	$13\frac{1}{3}$	1,068	802	913	Two of this group rebuilt in 1908. Built U. S.; good life, small repairs.
X	6	120x20x5	Pine with oak bottom.	1,300	22	15	21	2,871	443	2,098	All but one rebuilt.

MODEL BARGES.

Early in the improvement six oak model barges, 135 by 26 by 5½ feet, were built on the Ohio River, three by Howard, of Jeffersonville, Ind., and three by Cutting, of Metropolis, Ill. These barges, numbered 60-62 and 88-90, were built in 1882 at $3,500 each, and were not condemned until 1901, but for five or six years previous the repairs were very heavy. These barges were in use eighteen years.

QUARTER-BOATS.

The quarter-boats used in this improvement, in which category

Fig. 2. An office-boat and quarter-boat of the type most approved on the Upper Mississippi.

may be included office-boats and inspection boats, have been very numerous and always long lived, because it has been advisable to rebuild hulls or provide new ones on account of the cabins, which do not decay or wear out. The following table of selections shows this clearly. The dimensions and design of these boats have varied—in fact, it is believed that there are hardly any two alike.

QUARTER-BOATS. HULL, PINE OR FIR. CABINS, PINE.

No.	When built.	Dimensions. Hull. Feet.	Dimensions. Cabin. Feet.	Cost with outfit.	Material of hull.	Remarks.	Repairs and outfit to Dec. 31, 1910.	Condition, December 31, 1910.	Life. Years.	Designation.
75	1882	75x20x3	*60	**$700	Pine.	1882-1891 no repairs; hull rebuilt 1894 and 1907; large repairs 1898 and 1909.	$4,319	Fair	28	Q. B.
118	1891	70x20x3	*55	1,414	Pine.	Large repairs 1897, 1899, 1902, 1905, 1907; new hull 1910, fir.	3,205	Good	19	Q. B.
47	1894	75x20x3	60x19	1,561	Pine.	Nominal repairs to 1908 when hull was rebuilt.	1,628	Good	16	Q. B.
71	1894	75x20x3	60x19	1,138	Pine.	Nominal repairs to 1904; large repairs 1905-1907; hull rebuilt 1909.	2,763	Good	16	Q. B.
183	1895	100x20x3	80	2,698	Pine.	Nominal repairs to 1909, when hull was rebuilt.	3,199	Good	15	Q. B.
184	1895	60x18x3	45	871	Pine.	Nominal repairs to 1910, when hull was rebuilt.	1,558	Good	15	Q. B.
202	1895	70x20x3	55	1,328	Pine.	Nominal repairs to 1907, when hull was rebuilt.	2,269	Fair	15	Q. B.
11	1893	75x20x3	60x19	1,648	Pine.	Nominal repairs to 1910, when hull was rebuilt.	2,206	Good	16	Q. B.
65	1893	40x16x2	30x16	416	Pine.	Nominal repairs to 1907, when hull was rebuilt.	698	Good	16	Q. B.
94	1884	60x16x3	-------	250	Pine.	Small repairs to 1902, when hull was rebuilt; new hull in 1910.	1,429	Fair	26	Q. B.
91	1884	60x16x3	-------	452	Pine.	Small repairs to 1895, when hull was rebuilt; new hull in 1905.	1,400	Fair	26	Q. B.
121	1892	52x16x2½	40x12	430	Pine.	Small repairs to 1903, when hull was rebuilt; new hull in 1910.	1,197	Good	28	Q. B.
122	1892	52x16x2½	40x12	430	Pine.	Small repairs to 1903, when hull was rebuilt; new hull in 1910.	1,198	Good	28	Q. B.
123	1892	52x16x2½	40x12	430	Pine.	Small repairs to 1903, when hull was rebuilt; new hull in 1908.	848	Fair	28	Q. B.
124	1892	52x16x2½	40x12	430	Pine.	Small repairs to 1903, when hull was rebuilt; new hull in 1908.	1,027	Good	28	Q. B.
20	1897	64x18x3	50x18	1,452	Fir.	Small repairs to 1905, when hull was rebuilt.	982	Good	13	O. B.
67	1893	66x18x3	46x18	1,727	Pine.	Small repairs to 1907, when hull was rebuilt.	2,831	Good	17	O. B.
69	1893	50x16	38x16	607	Pine.	Hull partly rebuilt in 1904; small repairs in other years.	944	Good	17	O. B.

*A good part of the expense attached to these boats is in the renewal of outfit.
**The cost of No. 75 seems small, but it does not include outfit.

Building boats have not been standardized, although those recently built are quite similar. Many of these boats were adopted from ordinary barges. They are used in building dams, being suspended along the line of the dam; the brush and rock barges are handled with their power.

BUILDING BOATS.

Nos. 36 and 38, hand-power capstans; No. 64, steam power.

No.	When built.	Dimensions of hull. *Feet.*	Material of hull.	Cost.	Remarks.	Condemned.	Total repairs.	Good life.
36	1893	120x18x3	Pine	$1,385	Nominal repairs to hull and machinery to 1907	1908	$975	14
38	1893	120x18x3	Pine	1,385	Nominal repairs to hull and machinery to 1902. Bad from 1903 to 1908	1908	586	9
64	1895	160x26x4	Fir	3,786	Nominal repairs for four years; large repairs 1902 to 1908	1908	2,709	13

Hulls of building boats were not rebuilt, the capstans, etc., being transferred to new and improved hulls.

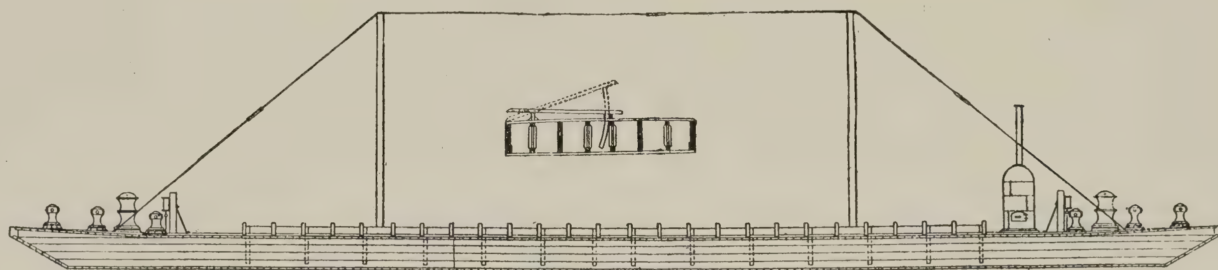


Fig. 3. Elevation and section of latest form of building barge (No. 64).

DUMP SCOWS.

Dimensions, 73 by 18 feet; eight pockets. Nos. 1 to 6, oak; Nos. 7 to 12, mostly fir.

No.	When built.	Cost.	Repairs.														Total repairs.	Good life.
			To 1891	1892	1893	1894	1895	1896	1897	1898	1899	1900	1901					
1	1885	$1,637	$690	$317	$98	Bad	Bad	Cond.	------	------	------	------	------				$1,195	8
2	1885	1,637	690	428	32	52	Bad	Cond.	------	------	------	------	------				1,202	8
3	1885	1,637	590	93	4	109	Bad	Cond.	------	------	------	------	------				796	8
4	1885	1,637	98	545	35	104	Bad	Cond.	------	------	------	------	------				782	8
5	1885	1,637	111	578	31	77	*1,359	------	------	138	------	------	Cond.				2,294	9
6	1885	1,637	555	149	30	Bad	Bad	Cond.	------	------	------	------	------				734	8
			1897	1898	1899	1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910		
7	1896	1,192	30	8	37	115	220	150	*687	2	43	7	116	209	388	Cond.	2,102	6
8	1896	1,187	30	19	52	63	217	171	Cond.	------	------	------	------	------	------	------	552	6
9	1896	1,625	4	9	49	115	79	150	Cond.	------	------	------	------	------	------	------	406	6
10	1896	1,651	3	6	36	79	215	152	102	171	110	Bad	Cond.	------	------	------	874	9
11	1896	1,651	------	------	------	2	28	338	*625	388	65	Bad	Cond.	------	------	------	1,446	9
12	1896	1,636	------	------	------	------	28	329	*664	401	107	Bad	Cond.	------	------	------	1,529	9

*Rebuilt. The rebuilding of No. 5 was not good policy. Nos. 7 and 8 used old irons. So much money for repairs on Nos. 11 and 12, 1902 to 1905, seems injudicious. The dump scows are of the usual side pocket type.

TOW-BOATS.

There are three sizes of tow-boats used in this improvement, which we designate as large, medium, and small. Of the boats mentioned in the following tables: The *Coal Bluff*, *Fury*, *Henry Bosse*, and *Alert*, are in the first class; the *Ruth*, *Mac* and *Grace* in the second; and the *Lucia*, *Louise*, *Elsie*, *Emily*, and *Ada* in the third. The *Elsie* was built with a steel hull, and the wooden hull of the *Louise* was changed to steel in 1905.

The *Fury* and *Henry Bosse* (formerly the *Vixen*) were built under contract at Dubuque, Iowa. Their hulls are of oak, 100 feet by 19 feet 6 inches by 3 feet 10 inches; cylinders, $10\frac{1}{2}$ inches

Fig. 4. Tow-boat *Henry Bosse.*

by 4 feet; one boiler, 22 feet by 42 inches, with ten 6-inch flues. Both of these boats have been rebuilt with somewhat different dimensions. On December 31, 1910, they were classed as fair, which means extensive repairs were needed.

The *Alert* was bought second-hand; hull, oak, 115 by 19 by 3 feet; cylinders, 10 inches by 5 feet; one boiler, 16 feet by 43 inches; rebuilt in 1884 and partially rebuilt several times. December 31, 1910, in bad condition.

The *Coal Bluff* was bought second-hand, 3 years old; hull, oak, 120 feet by 22 feet by 4 feet 6 inches; cylinders, 15 inches by 5 feet; three boilers, 26 feet by 36 inches; hull twice rebuilt and also very large repairs; condition, bad.

The *Mac* was bought nearly new; oak hull, 73 by 16 by 3 feet;

cylinders, 7 inches by 3 feet 2 inches; one boiler, 14 feet by 36 inches; hull has never been entirely rebuilt, although large repairs were made in 1894, 1902, and 1910; condition, good.

The *Ruth* was built by the United States; hull, oak, 75 feet by 17 feet by 3 feet 3 inches; cylinders, 7 inches by 4 feet; two boilers, 10 feet by 30 inches; hull has not been entirely rebuilt, but received large repairs in 1901 and 1909; condition, good.

The *Grace* was built by the United States; hull, oak, 79 by 17 feet; cylinders, 7 feet 6 inches by 4 feet 1 inch; two boilers, 10 feet by 30 inches; hull has not been rebuilt or received large repairs; condition, good.

TOW-BOATS (Large and Medium), OAK HULLS.

	Year.	*Fury*, built 1881.	*Henry Bosse*, built 1881.	*Albert*, built 1874; rebuilt 1881.	*Coal Bluff*, built 1878; rebuilt 1881	*Mac*, built 1891; rebuilt 1893.	*Ruth*, built 1895.	*Grace*, built 1904.
Original cost	------	$11,976.00	$11,976.00	$6,000.00	$8,000.00	$2,500.00	$6,426.47	$8,616.37
Repairs to	1890	6,292.71	10,432.01	*14,202.00	*25,769.09	------	------	------
Repairs	1891	‡3,187.09	‡2,032.26	133.32	------	------	------	------
Repairs	1892	426.35	588.95	409.31	‡3,628.02	------	------	------
Repairs	1893	1,973.82	375.52	‡1,586.61	‡2,236.53	653.06	------	------
Repairs	1894	255.93	749.85	‡1,770.12	1,283.67	‡1,379.93	------	------
Repairs	1895	†3,364.32	*5,050.51	559.08	702.83	362.34	------	------
Repairs	1896	2.90	272.45	590.88	215.86	539.22	99.44	------
Repairs	1897	463.70	232.30	‡1,938.49	1,028.62	430.38	159.28	------
Repairs	1898	497.46	200.42	‡1,034.95	72.72	571.35	231.57	------
Repairs	1899	*5,247.28	224.16	‡1,064.78	*9,122.30	400.07	289.95	------
Repairs	1900	147.19	‡1,744.01	191.28	602.44	434.25	397.82	------
Repairs	1901	38.91	231.22	------	223.05	142.09	‡1,112.67	------
Repairs	1902	998.34	629.91	304.47	1,175.51	‡1,862.03	275.00	------
Repairs	1903	495.76	460.44	‡1,619.06	277.86	237.65	338.59	------
Repairs	1904	461.53	523.90	476.68	621.19	195.54	799.11	------
Repairs	1905	583.63	‡3,375.25	692.31	1,612.76	874.04	220.38	119.33
Repairs	1906	1,209.11	61.12	‡1,208.05	858.65	210.01	643.98	28.86
Repairs	1907	‡3,026.34	87.41	‡1,804.35	‡3,204.10	457.66	492.41	225.79
Repairs	1908	337.25	497.52	‡1,574.92	‡3,226.92	631.64	661.29	35.65
Repairs	1909	719.70	695.27	‡1,680.45	280.74	606.00	‡2,881.30	266.00
Repairs	1910	1,178.67	1,210.28	‡2,720.89	‡1,686.93	‡1,927.61	766.30	157.67
Totals		$42,883.89	$41,650.76	$41,562.00	$65,829.79	$14,414.87	$15,795.56	$9,449.67

*New hull. †New boilers. ‡Very large repairs to hull.

The total cost of each of the first three boats mentioned in this table appears very small for thirty years service and while considering the greatly increased cost of lumber, it may now be advisable to build of steel or iron, it is manifest that wooden hulls in the past were cheaper than metal would have been.

SMALL TOW-BOATS.

The *Lucia* was built by the United States at Keokuk; hull, oak, 68 feet by 12 feet 8 inches by 3 feet; cylinders, 6 inches by 2 feet 6 inches; boiler, 10 feet by 38 inches. She had large repairs in 1892 and 1904, and her hull was rebuilt in 1895 and 1909-1910; condition, December 31, 1910, good.

The *Louise* was built by the United States at Keokuk; hull, oak, 61 by 12 by 3 feet; cylinders, 6 inches by 2 feet 6 inches; boiler, 10 feet by 34 inches; hull rebuilt in 1894; steel hull in 1905; moderate repairs each year; condition, good.

The *Elsie* has a steel hull and was built by contract at Jeffersonville, Ind.; hull, 67 by 13 by 3 feet; cylinders, 6 inches by 2 feet 6 inches; boiler, 10 feet by 34 inches. The *Elsie* appears to have cost as much money as the wooden hull *Ada* for the same period of time.

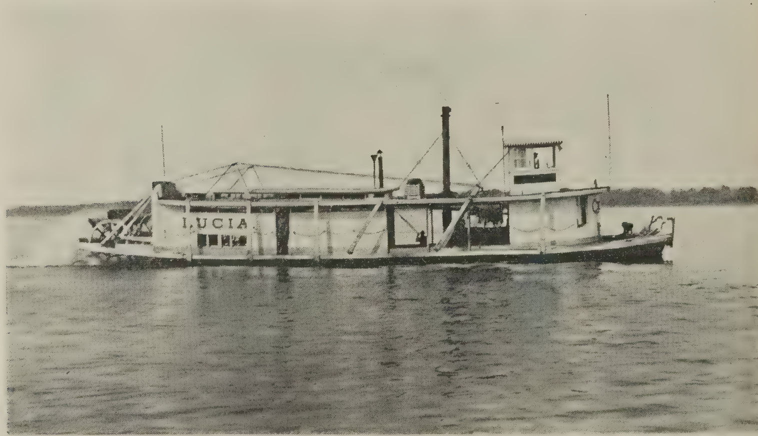


Fig. 5. Small tow-boat *Lucia*.

The *Emily* was built by the United States at Keokuk; hull, oak, 67 by 12 by 3 feet; cylinders, 6 inches by 2 feet 4 inches; boiler 10 feet by 34 inches; condition, good; new hulls in 1902 and 1909-1910.

The *Ada* was built by the United States at Keokuk; hull, oak, 68 by 11 by 3 feet; cylinders, 6 inches by 2 feet 6 inches; boiler, 10 feet by 34 inches; condition, good; hull rebuilt 1903-1904.

These small tow-boats are of great value with light tows and in working around the dams.

TOW-BOATS. (Small.)

	Year.	*Lucia*, built 1895.	*Louise*, built 1884.	*Elsie*, built 1889.	*Emily*, built 1889.	*Ada*, built 1889.
Original cost	----	$4,000.00	$3,538.00	$5,110.00	$4,034.00	$4,000.00
Repairs to and including	1890	1,560.62	761.25	194.89	175.28	112.70
Repairs	1891	27.79	221.47	200.58	17.57	37.29
Repairs	1892	*a*1,181.67	--------	21.91	350.55	16.55
Repairs	1893	152.45	527.41	519.80	60.82	593.18
Repairs	1894	296.10	*3,010.99	387.87	154.74	791.11
Repairs	1895	*2,286.84	399.96	619.02	328.06	730.04
Repairs	1896	331.57	333.86	102.63	48.25	262.94
Repairs	1897	137.51	84.67	227.11	854.42	475.93
Repairs	1898	58.11	96.22	534.62	55.54	557.22
Repairs	1899	142.25	60.10	112.64	86.74	142.20
Repairs	1900	78.64	565.27	35.52	166.64	47.15
Repairs	1901	87.73	--------	1.26	--------	--------
Repairs	1902	156.02	323.20	319.63	*2,908.74	394.54
Repairs	1903	75.07	349.21	87.42	12.10	*1,045.64
Repairs	1904	a1,086.20	259.12	751.60	103.56	*1,583.06
Repairs	1905	44.51	*b*2,991.17	266.49	205.28	50.60
Repairs	1906	80.52	326.60	194.43	82.49	136.75
Repairs	1907	453.22	368.42	583.86	410.16	328.19
Repairs	1908	186.60	212.91	807.72	447.80	127.21
Repairs	1909	*1,107.44	62.47	331.29	*850.31	364.00
Repairs	1910	*3,044.29	541.18	1,150.08	*3,123.56	454.30
Totals		$15,485.15	$15,033.48	$12,560.37	$14,476.61	$12,250.60

*New hull built. *a*Large repairs. *b*New hull built. Steel.

All of these boats, except the *Elsie*, had wooden hulls when built. The *Elsie's* hull is steel and the *Louise* has also a steel hull since 1905. The *Elsie*, *Emily*, and *Ada* were built in the same year, and the cost of the two latter compares favorably with the former.

DREDGES.

The statement below includes three dipper dredges, *Ajax, Vulcan,* and *Phoenix,* and two pipe-line dredges, *Geyser* and *Hecla.* As will be noted, the care and upkeep of dredges are very expensive, and in the case of suction dredges the pontons and catamarans also require much repair.

The *Ajax* has hull dimensions, 70 by 26 by 6 feet; she was re-

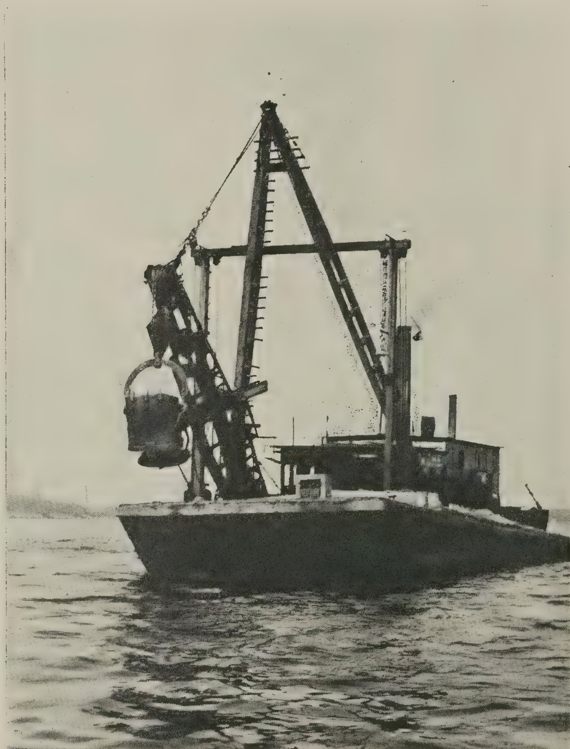


Fig. 6. Dipper dredge *Vulcan.*

built in 1894 and, with large annual miscellaneous repairs, has been kept in good condition.

The *Vulcan,* oak hull, 80 by 30 by 8 feet; nominal repairs to 1890; hull rebuilt in 1892-1893 and 1898-1899; condition now, good, although annual repairs have been large for the past eight years.

The *Phoenix,* oak hull, 80 by 30 by 8 feet; nominal repairs to 1890; hull rebuilt in 1895-1896; burned and entirely rebuilt,

	Year.	Dipper Dredge *Ajax*, bought 1877; built 1876. (Oak hull.)	Dipper Dredge *Vulcan*, built 1882-1883. (Oak hull.)	Dipper Dredge *Phoenix*, built 1885. (Oak hull.)	Suction Dredge *Geyser*, built 1893. (Fir hull.)	Suction Dredge *Hecla*, built 1900-1901. (Fir hull.)
Original cost	----	$11,300.00	$19,450.00	$19,525.00	$4,704.44	$27,708.66
Repairs to and including	1890	--------	2,840.82	2,848.27	--------	--------
Repairs	1891	--------	--------	199.28	--------	--------
Repairs	1892	a11,539.60	*1,525.61	797.78	--------	--------
Repairs	1893	536.03	*3,373.06	766.71	--------	--------
Repairs	1894	*5,801.75	27.92	391.34	2,032.36	--------
Repairs	1895	1,494.86	1,337.78	*1,582.93	1,661.20	--------
Repairs	1896	713.00	1,904.42	*3,033.18	639.33	--------
Repairs	1897	1,177.85	903.49	806.32	1,299.48	--------
Repairs	1898	423.26	*2,256.89	1,166.02	1,462.74	--------
Repairs	1899	1,079.07	*3,887.66	1,123.26	1,344.31	--------
Repairs	1900	1,029.55	614.45	1,360.17	1,646.45	--------
Repairs	1901	490.69	349.59	931.50	1,177.04	--------
Repairs	1902	747.92	822.36	889.86	1,266.11	1,081.41
Repairs	1903	1,425.36	1,530.67	306.70	1,905.00	3,405.09
Repairs	1904	449.03	2,223.93	999.13	1,040.71	3,395.78
Repairs	1905	709.33	1,955.35	903.92	2,793.23	3,650.24
Repairs	1906	2,646.86	2,546.13	1,396.33	2,171.35	1,677.00
Repairs	1907	1,709.79	1,745.82	1,818.27	1,960.88	2,681.56
Repairs	1908	1,210.24	2,837.16	b7,859.91	3,512.48	1,044.20
Repairs	1909	1,829.54	2,986.84	b11,721.38	2,713.74	2,692.51
Repairs	1910	1,788.47	1,996.41	1,674.71	2,897.12	1,421.59
Totals to date	----	$48,102.20	$57,116.36	$62,101.97	$36,227.97	$48,758.04

*New hull, etc. *a*Repairs to December 31, 1892. *b*Burnt and rebuilt.

using a portion of the old machinery, in 1908-1909, at a cost of \$19,581.29; now in good condition.

The *Geyser*, with eleven pontons, built by the United States at small cost, using old boiler and pump; hull, pine, 100 by 20 by 4 feet; pump, 12-inch suction; large expenditures each year for pump, pipe pontons, etc., in addition to hull repairs; condition, bad.

The *Hecla*, 15-inch suction dredge, with eleven pontons, built by United States; large repairs every year; hull, fir and oak, 120 by 26 by 5 feet; rebuilt, 1909-1910; good condition.

Acids in Rivers from Mines and Mills, with Special Reference to the Monongahela

BY

Mr. THOMAS P. ROBERTS*
Assistant Engineer

[Condensed, by Mr. Roberts, from his article on the subject and the discussion of same in the Proceedings of the Engineers' Society of Western Pennsylvania, Pittsburgh, November, 1912.]

The object of this paper was the promotion of efforts to mitigate the evils resulting from the unrestricted use of water courses for carrying away harmful fluid discharges from mines and mills and the presentation of some observations on the phenomena attendant upon acids in the Monongahèla River.

The evils referred to have reached, in the Monongahela, such vast proportions in recent years as to bid fair to render it unfit for the navigation of the ordinary type of steamers, and very costly for power plants using the river water for making steam.

The author suggested as one expedient, easy of application, the stoppage of the custom at some mills of discharging "pickling vats" with a rush into the river, thus developing local areas of high acidity. Steamers seeking landings, with their boiler pumps in operation in such areas, will take up more acid in a brief period than they would during many hours while under way in mid-river.

Owners of steel mills have never, it appears, made any concerted effort to abate the acid nuisance in the river water, but have generally chosen instead to invest large sums of money in the erection of softening plants where the water, before it is permitted to reach their boilers, is treated usually with soda ash and lime to neutralize and soften it. There are examples of companies, owning a number of mills, letting acid into the river from one plant, which is bound to increase the difficulties at another of their own plants farther down the Monongahela. The cost of operating softening plants is very considerable, being several hundred dollars daily in some instances.

* * * According to the investigations of Mr. Charles E. Ashcraft, Junior Assistant United States Engineer, Pittsburgh office, much the largest proportion of the sulphuric acid reaching

*Pittsburgh Engineer District.

the river comes from coal mines. He has ascertained that of the twelve galvanizing establishments on the Monongahela River, or its tributaries, eight of them are estimated to use 59,200 tons of acid per annum. The four plants not included in his estimate are very extensive users, and it is probably within the mark to say that 75,000 tons of acid are annually consumed in the district. One of the establishments finds a profit in converting its spent acid into copperas, while several others waste but little acid. At one establishment the waste acid reaching the river amounted to about 6 per cent of the annual consumption. Data are lacking as to per cent of waste for a number of the more important plants. On the assumption that the waste acid from the twelve plants reaching the river forms 2 per cent of their total consumption, the amount would be about 4 tons daily reaching the river.

With the development of mining operations, especially in West Virginia, and the erection of manufacturing plants in recent years, there are times when the entire river is artificially clarified by the acid as far as Fairmont, 130 miles above Pittsburgh. As nearly as can be estimated, there appears to be constantly in the river at Pittsburgh enough acid to make a showing of clarification with a discharge represented by 3 feet depth on a dam 900 feet in length. This would make a volume of water approximately 15,000 cubic feet, or 112,500 gallons, per second.

It appears that while the records of tests for acids in the river available for study are numerous, covering, as they do, daily reports from mills and pumping plants for a number of years past, nevertheless the records have not been coordinated with the river's discharge.

During September, last, immediately following a freshet and before time had elapsed sufficient for the concentration of acids about galvanizing plants to develop, and while the river's discharge was 112,500 gallons per second, Mr. Roberts had five samples of the water analyzed. But little difference was found between that flowing in mid-river (width 1,000 feet) and that near the shore, the mean of the samples indicating .093 grain of free sulphuric acid per gallon. This would make 64.5 tons of acid daily. There is reason to think, according to Mr. Roberts, that this 64.5 tons represents the mean daily movement of acid at Pittsburgh for most of the year. For months at a time, however, during some seasons the total discharge of the river at Pittsburgh may not average more than 2,250 gallons per second, so that with the same quantity of acid its strength at such periods would be represented by, say, 4.65 grains per gallon.

Some of the side streams, such as Turtle Creek, discharge water into the river carrying from 8 to 20 grains of acid per gallon, while from some mines the effluent water is loaded with from 200 to 500 grains per gallon. (As against this, as may be gleaned from the discussion of the author's paper, some mines discharge strong alkaline compounds, so that there may be acid, neutral, and

alkaline areas. The author dealt only with the final "blend" of the water, so to speak, reaching Pittsburgh.)

The author stated that the action of the acid in the water, at river temperatures, begins to manifest itself on steel or iron when there may be somewhat less than 1 grain per gallon. In boilers, in addition to a higher temperature, the acid rapidly concentrates, hence the necessity for all steamboats on the Monongahela to use softening compounds. About the locks plate valves, etc., especially where strong currents strike them, rapidly become corroded. The author displayed numerous slide views showing valves, pipes,

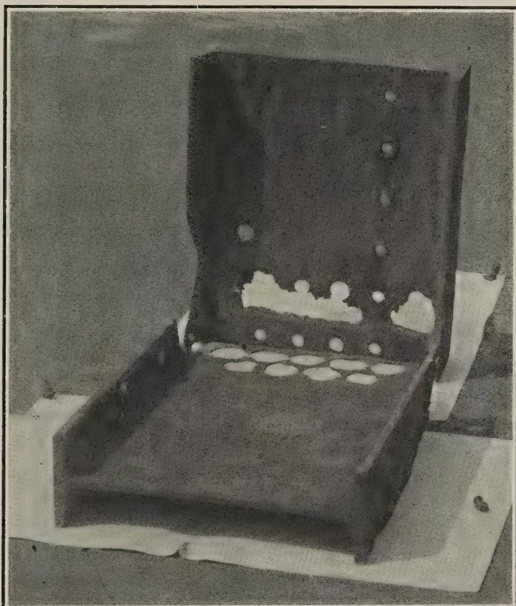


Fig. 1. Ends of I-beams from lock gates.

eyebeams, angles, and riveted work greatly damaged, so as to be beyond repair. One of the most interesting of these exhibits was the case of an angle bar, ¾-inch thick, from a miter sill, with vertical leg 8 inches and horizontal leg 6 inches, where, after less than a year's usage under pressure of full lock with acid in the water averaging about 2 grains per gallon, the vertical leg was entirely consumed, "sheared off," as it were, from the horizontal leg, while the latter had suffered but little loss of material on account of it not being in the field of powerful currents.

* * * Already one of the railroads paralleling the river has extended a pipe line from a reservoir in the mountains to meet the demands of its service. But, as the conduit from the great Ashokan Reservoir to New York City would scarcely have capacity suf-

ficient to meet the requirements of the manufacturing and furnace plants along the Monongahela, a water supply for them from the mountains is beyond reasonable expectation. The industrial establishments and the communities along the bank of the river, which does not include the city of Pittsburgh, according to records in the United States Engineer Office at Pittsburgh, in 1908 demanded a supply of 494,000,000 gallons daily, not all of which, however, requires treatment before use. The author estimates that at this date, including the tributaries of the Monongahela, there is pumped more than 600,000,000 gallons daily, a quantity equalling more than three times the ordinary summer discharge of the river. The quantity of water withdrawn from the river and lost in steam, or otherwise evaporated, is estimated at approximately 42,000,000 gallons daily.

The author's paper covered 16 pages of the proceedings, and touched upon a number of curious and interesting phenomena to be observed in the river from time to time. Reference was also made to the destruction of fish life and to the losses to domestic users of the river water caused by the acid, which can not be filtered out, which losses—added to cost of maintenance of softening plants at the steel works, etc.—must, in the aggregate, represent an enormous sum of money annually. Finally, the author adds "must steamboats be equipped with condensing engines and thus be unduly weighted, or must they take in tow bulk boats containing Allegheny River water, as some of them do, to draw upon for steam making? The situation is only bearable now at a large cost annually for repairs." * * *

By invitation of the society, several specially qualified engineers and chemists were present to take part in a discussion of the paper, which, as will be noted in the following abstracts, covered almost every phase of acid pollution from mines and mills along with the difficulties in the way of abating the evils complained of.

Discussion.

Mr. J. R. Campbell
Chief Chemist, H. C. Frick Coke Company
Scottdale, Pa.

Mr. J. R. Campbell, the first speaker, presented some interesting tables exhibiting composition of mine water, also developing the possibility of pigment production from them, and in part said:

For the benefit of any present who are not familiar with the chemical composition of mine drainage, we would call attention to the following analyses, the first two of which represent samples taken from our own coke operations, and the third a sample from a plant of the Pittsburgh Coal Company.

Examples of Mine Water.

	Grains per United States gallon.		
	L.	*S.*	*J.*
NaCl	1.75	2.59	57.73
Na_2SO_4	13.12	10.34	67.56
$CaSO_4$	86.30	32.05	77.28
$MgSO_4$	56.54	22.09	18.29
SiO_2	2.10	2.33	1.52
$Fe_2O_3+Al_2O_3$	----------	51.54	----------
Fe_2O_3	12.07	----------	45.50
FeO	72.53	16.50	(Not det.)
Al_2O_3	6.65	----------	116.05
SO_3	93.23	93.44	SO_3
H_2SO_4 (free)	26.30	213.50	1013.46

Note: The figure 1013.46 in J is the SO_3 that is both free and combined with the iron, the greater portion being free SO_3.

Water Pumped Daily.

	L.	*S.*
Maximum	1,500,000	15,000,000
Minimum	750,000	5,000,000
Free SO_3 (Min.)	1 ton (Max.)	180 tons
CaO for neutralizing SO_3	.7 ton	126 tons
Cost per day	$3.50	$630.00

* * * All present are perhaps familiar with the origin of "free acid" in mine drainage—that is, the oxidation of the iron pyrites (FeS_2) in coal by means of air and water through the various stages of ferrous sulphate, ferric sulphate, etc., until the acid radical is entirely free, forming the free acid, which gets to be of enormous proportions in the case of an old mine working.

We wish to point out, in the case of examples cited, some of the commercial aspects of the question of mine drainage. We will not consider sample J, since it is an old and an abandoned working. In samples L and S there are 26.3 and 213.5 grains of sulphuric acid, respectively, and taking the minimum pumpage daily at mine L, with the minimum acid in it, and the maximum pumpage at mine S with the minimum acid in it, and the maximum pumpage at mine S with maximum acid in it, we have some very interesting figures.

In the first case we find that there is discharged in twenty-four hours 1 ton of free SO_3, and in the second case 180 tons of free SO_3 in the same time. This would, perhaps, represent the maximum and minimum of any condition we might have.

Obviously the simplest and cheapest method of chemical treatment is to neutralize the "free acid" with lime, which we have taken at a cost of $5.00 per ton. In the case of mine L the daily cost of neutralizing the "free acid" would be $3.50, and in mine S $630.00. The increased cost per ton of finished product, at the above rates, is 3½ mills per ton in the case of mine L and 30 cents per ton in the case of mine S, which, in these days of close competition, would almost put a concern out of business in the last case cited. * * *

* * * But, so far as we know, there has been no practical method found for separating the iron oxide from mine water, even though it precipitates readily by the addition of heat, or the proper chemical. We know of one concern which is willing to buy sulphur mud, such as is found along water courses, sumps, and gob sections, where it often deposits to the depth of several feet. This concern has offered us as high as $4.00 a ton for sulphur mud, or the deposit from mine water, even when badly contaminated with impurities. By referring to the analyses of this raw and finished material, it will be noted that the iron oxide content is very high, ranging up to 95 per cent.

It may be that the time will come when the iron oxide in mine water will come to be a very valuable by-product and that operators can then afford to neutralize the "free acid" at the same time recovery of the paint pigment is made, which would be a happy solution of the problem.

Mr. R. B. Dole
Assistant Chemist, United States Geological Survey
Washington, D. C.

Mr. Dole, referring to a large coal field map of Pennsylvania and West Virginia, spoke in part as follows:

I wish to call attention to a few figures showing the extent of the pollution, and the necessary magnitude of any plans to abate it. These figures prove that the question of mine drainage far exceeds in importance the question of pickling liquors from galvanizing works and similar manufactories.

The shaded portions of Fig. 7 indicate the areas of coal, according to reports of the United States Geological Survey. These coal veins of Pennsylvania and the contiguous layers contain sulphur in several forms, possibly the most important of which in this connection are the sulphides of iron, which, becoming oxidized, form sulphuric acid that is in turn capable of dissolving iron and other bases, thus forming sulphates in solution. Without going into the theoretical reactions, it may be said that an excess of free sulphuric acid is nearly always left and, consequently, free sulphuric acid exists in the drainage from the mines to the streams. When the corrosive solution reaches the streams from the mine it

decomposes the carbonates and bicarbonates in the river waters, replacing them by sulphates. If there is enough sulphuric acid to exceed the weaker acid radicals, free sulphuric acid exists in the main stream. If not, the sulphates are increased at the expense of the carbonates, but the river water is still slightly alkaline. This disturbance of the percentages of carbonates and sulphates in the mineral residues obtained by evaporating the river waters to dryness affords a basis for estimating the extent of the sulphuric acid pollution. * * *

* * * Wherever mine drainage in any considerable quantity reaches a stream the percentage of sulphates is increased and the percentage of carbonates is decreased. * * *

* * * It is a simple arithmetical problem then to calculate

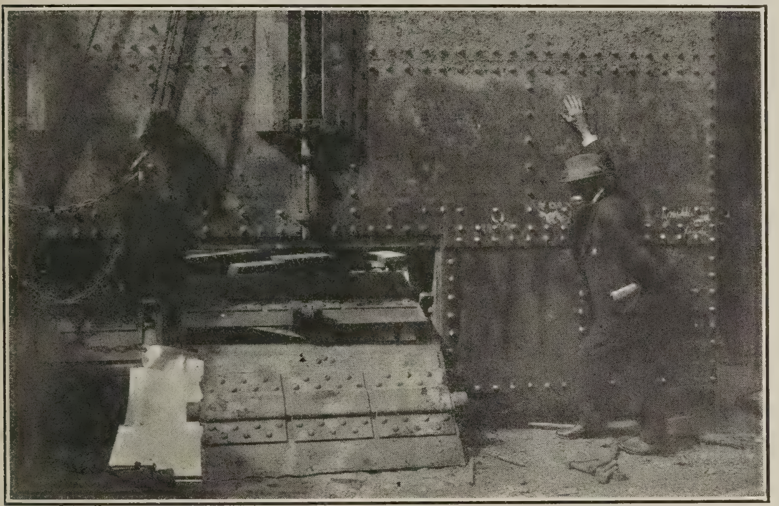


Fig. 2. Gate wicket showing loss of several inches of metal, ¾ inch thick, at bottom of blade by corrosion.

that the excess of sulphates in ten Pennsylvania streams give a total of 970,000 tons a year of acid. In other words, the sulphur in the coal in the drainage basins of these few streams produces annually more than half as much sulphuric acid as the listed production of the entire United States. When we consider that this large figure represents only 26,000 square miles of territory, we can readily believe that the total production of acid from the mines of Pennsylvania is not less than a million and a half tons a year. All this acid does not exist in the streams in the free state nor does it reach the streams always in the free state, for part of it forms soluble salts with iron, limestone, and other substances, and thus becomes partly neutralized. But the figure represents the magnitude of the original production of acid from the sulphur in the coal. * * *

Mr. Dole, after referring to great cost of entirely abating the acid nuisance, says:

* * * Thus it can readily be seen that while compulsory treatment of the waste before it reaches the streams is not absolutely out of the question, it is not at all a small problem or one that can be solved without enormous expense.

* * * The enormous quantity of this waste and its commercial value in concentrated form might well bring it into attention as a research problem for industrial chemists. * * * It is worthy of note that in a neighborhood where nearly a hundred thousand tons a year of acid are consumed in factories, one stream (Youghiogheny) carries away every year more than a million dollars worth of acid produced by nature.

Mr. W. E. Snyder

Mechanical Engineer, American Steel and Wire Company
Pittsburgh, Pa.

Mr. Snyder, after referring at considerable length to the consumption of acid at several mills with which he was connected, spoke of a certain coal mine which contributed much more acid to the river than that wasted from all the mills along the Monongahela, said as follows regarding the destruction of fish and other life forms:

* * * I am too much of a lover of nature and of all kinds of wild life to close this discussion without some reference to the esthetic side which has been incidentally mentioned in the paper—that is, the harmful effect of the pollution of the water upon all kinds of fish life that is found in rivers having pure water; but again, I would call attention to the effect of the mine water on streams on which there are no manufacturing plants whatever. There are numbers of small streams in this section of the State on whose watersheds there are no manufacturing plants, and yet all kinds of life have practically vanished from the waters of these streams after the opening of the coal mines on their watersheds. * * *

In closing he said:

* * * Mining and using Pittsburgh coal evidently levies an assessment on all the people of this District, whether they know it or not.

Mr. E. C. Trax

Chief Operator, Municipal Filtration Plant
McKeesport, Pa.

The paper of the evening is a valuable addition to the literature of this subject, the importance of which is fast becoming realized. The increasingly large amount of free sulphuric acid and sulphate

of iron in the streams of this district is indeed becoming a serious problem. The experience of the city of McKeesport with acid water, previous to the installation of the treating plant, is pretty well known; the fact that brass valves are rapidly eaten away, and even the "acid proof" bronze impellers of centrifugal pumps affected to some extent, indicates the corrosive action of the Youghiogheny River water. Engineers from other sections of the country are appalled when the condition of this stream is brought to their attention. During 1910 a new mark was set for extreme acidity; on October 30 the acidity to phenol-phthalein was 35 grains per gallon, and to methyl orange, which is a close approximation of the free sulphuric acid, 23 grains per gallon.

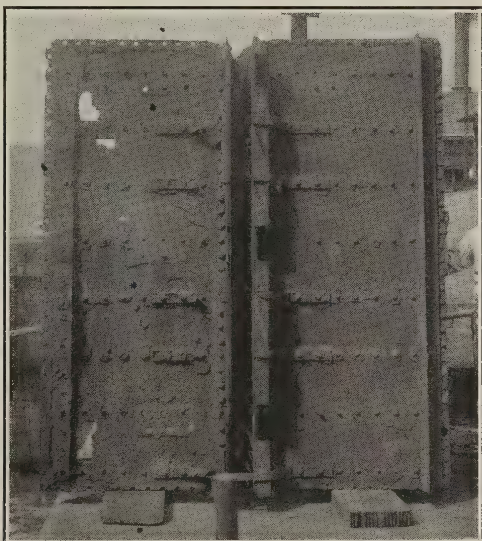


Fig. 3. Butterfly valve corroded beyond repair.

The disadvantages of the acid in relation to water purification for domestic purposes are generally summed up by high cost of treatment; this includes the necessity for more expensive construction of treating plants, high cost of chemicals, additional expense of labor for handling chemicals, wear and tear on machinery, etc. The purification of water containing acid iron wastes presents no special difficulty, except that of cost. There are, indeed, several features which have even been considered slight advantages, although they might well be overlooked in comparison with the array of injurious effects. * * *

* * * The following table of monthly averages of acidity and number of bacteria in the Youghiogheny River at McKeesport, shows the effect of the varying acidity on the germ life.

Month.	Av. acidity* Grains per gallon.	Bacteria per cub. centimeter†	
		24 hrs. incub. at 37° C.	48 hrs. incub. at 20° C.
January	0.64	576	31,000
February	2.16	105	20,000
March	2.10	2,000	21,000
April	3.03	446	12,000
May	1.34	178	2,000
June	0.35	681	6,500
July	3.79	167	205
August	10.62	65	9
September	6.59	682	97
October	14.00	300	240
November	10.27	416	160
December	1.69	98	2,400

*Acidity to methyl orange. Average of three to eight determinations made each day.

†Average of daily determinations.

Mr. J. N. Chester

Civil and Sanitary Engineer, Chester & Fleming
Pittsburgh, Pa.

According to Mr. Chester our Creator put navigation of our rivers last in His scheme of development of the human race, according to which creed Congress had better be on the lookout to find out whether there is anything to be left for it to do on some of the western rivers. He said in part as follows:

I believe if we could look into the purpose of our Creator in giving us the streams, we would find they were primarily put there for drainage purposes, and that is the first benefit creation got from them. The next thing was as a water supply, and probably after that came means of transportation. The initial paper to-night would protect navigation to the exclusion probably of everything else. Constituted as many of our bodies are that have charge of the streams, there seems to be foregathering a conflict of authority. Already we have the navigable streams in charge of the War Department and, primarily, the United States engineers. In this State we have the purity of the streams to a certain extent guarded by the State board of health, and we have still another authority looking after the streams to a certain extent, in the way of the State water commission. In these three bodies there is a conflict of interests.

Mr. Morris Knowles
Civil and Sanitary Engineer
Pittsburgh, Pa.

Mr. Knowles, referring at some length to "possible benefits due to acids" and to "injurious effects of acids," on which topics he had been requested to speak, said in part as follows:

It has been stated by the author that the acid has some sterilization and clarification effect upon the river water. It seems to be a somewhat popular notion that the benefit of this action may be relied upon to the exclusion of other well-known remedies for bad water. It is true that an acid condition of water will destroy organic and bacterial life, and thus render the use of such water less harmful from the hygienic standpoint at this time. It has been noticed in recent years, even before the introduction of filtered

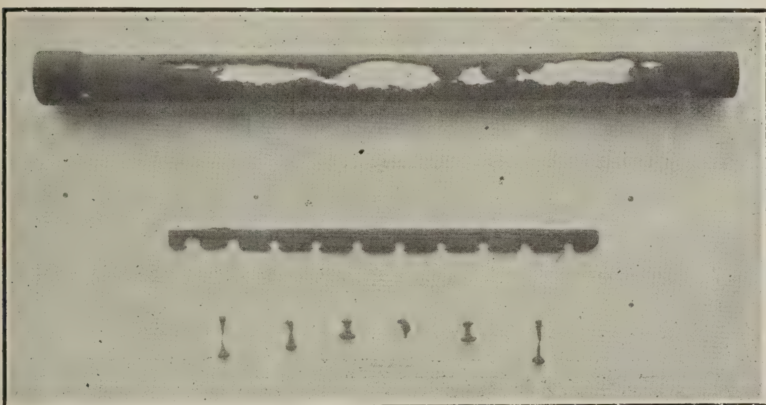


Fig. 4. Corroded pipe, rivets, etc.

water upon the South Side, that at certain times when the Monongahela was low and acid, the bacterial content of the water was much less than would naturally be expected from the large amount of sewage received from the populated territory above. It is even stated that typhoid fever was less prevalent upon the South Side than in the remainder of the city. It is possible that some of the improvement effected may be due to the extensive heating of a portion of water in the various steel mills and return of this amount to the river in a sterile condition.

* * * Neither the sterilization nor the clarification, which is due to the presence of acid wastes in a river water, can be depended upon to safeguard the public health. One can never tell whether either of these effects will be obtained at any given time. In any water improvement system uniform action is absolutely necessary, and reliance should not be placed upon any phenomena which may or may not occur.

* * * It has been found that an acid condition of the Allegheny River may destroy the bacterial film on the surface of a slow sand filter, thus permitting the water to pass through the filter more rapidly than it should and without successfully purifying. The important function of the principal feature of the filter is therefore rendered non-effective, so that dependence upon that which is expected to safeguard health is useless; again, we see the acid condition of our streams is really injurious.

* * * It appears that the most important thought which can be obtained from the paper is that the time is now ripe to thoroughly study this perplexing subject. We all realize that the presence of acid in the streams is injurious and harmful, even in a diluted condition. There are few data of a concordant kind relative to the variations in acid content with the height of stream and the volume of flow. A great deal of good would result if all of this information were collated and brought together in one complete statement.

* * * Legislation, however, is of little effect unless backed by a strong general sentiment; laws which are far in advance of public opinion can not be enforced. It would do no good at the present time to pass drastic laws preventing acid entering streams. Granting the possibility, such acts would materially increase the cost of production of important items of manufacture and household use and would add to the cost of living. The problem of improving the present conditions is a serious one, and thoughtful study will be required to find the best remedy and whether the mines and mills can mitigate the situation without an enormous expense.

Mr. J. C. Wm. Greth

Manager, Water Purifying Department, Wm. B. Scaife & Sons Co. Pittsburgh, Pa.

Mr. Greth, referring to mine drainage of alkaline water, spoke as follows:

In speaking of mine drainage it should be borne in mind that many mine waters are not acid but decidedly alkaline and that such waters tend to mitigate the evils arising from the discharge of acid waters, both on account of the dilution and the reactions between the various substances in solution, neutralizing the free sulphuric acid and acid salts of the river water.

Mr. Campbell cited as an exceptional case one mine water, from an abandoned mine, in which the free acid content was 1000 grains per United States gallon (less than 2 per cent solution). The average of the total acids and acid salts in mine drainage will not exceed 100 grains per United States gallon (less than a two-tenths of 1 per cent solution).

To neutralize the acids of mine drainage by the use of chemicals, of which limestone is the cheapest, assuming drainage of average

acidity, would add from 3 to 10 cents per ton to the cost of mining coal, which the coal operator would consider prohibitive. Lime can also be used, but would still further increase the cost of neutralizing the acids, etc.

* * * The seepage from abandoned mines presents another phase to the problem of preventing stream pollution. Water from these old mines contains much more free acid and acid salts than at the time the mine was being worked. To place the responsibility for contamination from this source opens up a rather difficult question to be decided.

* * * The comparative quantities of reagents required to soften water for different stages and seasons can readily be obtained from the curve. These curves* will show a variation in the soda ash required from two-tenths of 1 pound per 1000 gallons to nearly 2 pounds, and the lime from less than two-tenths of 1 pound to eight-tenths of 1 pound per 1000 gallons, or a variation in cost of treatment of from less than one-half cent per 1000 gallons to over 3 cents per 100 gallons—based on lime at one-half cent per pound and soda ash at 1¼ cents per pound.

Mr. C. A. Finley
Superintendent, Bureau of Water
Pittsburgh, Pa.

Mr. Finley presented much information concerning acid and alkaline conditions of the Allegheny River above Pittsburgh, many tributaries of the river being already highly contaminated from mining operations.

* * * Already some of the tributaries of the Allegheny have been materially affected by mine waste, mill waste, or a combination of the two. Most notable of these are the Kiskiminetas, the Conemaugh, the Loyalhanna, and Black Lick. These streams are known to receive the mine drainage from operations of considerable magnitude. Many Pittsburghers remember the Kiskiminetas Creek as very attractive fishing grounds. For several years past, however, the stream has been entirely devoid of fish, on account of the acid condition of the water.

The appearance of the water at the junction of the Allegheny and the Kiskiminetas at Kiskiminetas Junction is generally very striking. There is a very distinct line of demarcation between the brown, alkaline water of the Allegheny and the blue or green acid water of the Kiskiminetas.

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*Curves not submitted.—Ed.

Mr. James O. Handy
Chief Chemist, Pittsburgh Testing Laboratory
Pittsburgh, Pa.

Mr. Handy, after explaining the development of acids from coal mine water and how there is occasionally a natural naturalization of the water, advanced with his subject as follows:

It is remarkable what damage very small amounts of acid will cause in steam boilers. One of my first pieces of work when I came to Pittsburgh was the investigation of the amount of free acid in the Monongahela River water near the Carrie Furnaces. Some of the samples were not acid at all and others only contained very small amounts of acid. It seems to be a fact that sulphuric acid and sulphate of iron, when put into steam boilers, start a sort of perpetual motion operation. The acid attacks the iron and the iron salt which is formed is broken up; then the acid acts over again. There is constantly increasing accumulation of Venetian red or iron oxide in the boiler and pitting or perforation of the boiler goes on apace.

I give at this point analyses of mine waters, water from coal washing plants and waste pickling acids from steel mills.

Analyses of Mine Waters; Parts per 100,000.

	Mine K. H.	Mine K. S. 5,000,000 gals. per day.	Mine J. 2.
	Parts.	*Parts.*	*Parts.*
Total acidity	375.8	408.17	77.42
Ferrous iron	1.5	3.00	.50
Ferric iron	101.5	105.50	15.00
Alumina	41.0	29.00	15.86
Free sulphuric acid	------------	42.57	------------

Analysis of Water from Coal Washer K. S.

Total acidity	5.88 parts per 100,000
Ferrous iron	3.50 parts per 100,000
Ferric iron	1.50 parts per 100,000
Alumina	none.
Free sulphuric acid	none.

Waste Pickling Acids; Parts per 100,000.

	J. A. Wire mill.	A. M. Sheet mill.
Total acidity	24,500	20,825
Ferrous iron	13,750	9,550
Free sulphuric acid	437	*4,112

*Equivalent to 2,420 grains per gallon.

Mr. B. A. Ludgate
Assistant Engineer, Pittsburgh and Lake Erie Railroad
Pittsburgh, Pa.

Mr. Ludgate presented some interesting tables covering several years' records of acid tests on the Youghiogheny and Monongahela rivers, and said:

The Pittsburgh and Lake Erie Railroad has an investment of $100,000 for water softeners on the Youghiogheny and Monongahela rivers above McKeesport, and the yearly cost of chemicals (soda and lime alone) for the fully treated water is $5,000.

In past years there has been comparatively more mining done

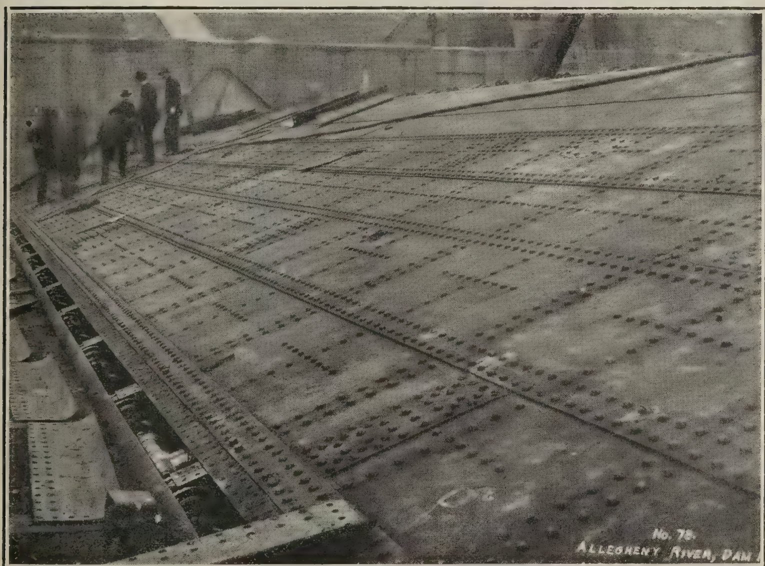


Fig. 5. Lower leaf of a ''bear-trap'' gate in course of construction.

along the Youghiogheny than along the Monongahela River, and from now on the amount along the Monongahela River is being increased very much, and I understand from Mr. R. R. Hice, State Geologist, that the amount of iron pyrites in the coal fields of West Virginia, tributary to the Monongahela River, is nearly twice as much as in the coal fields tributary to the Youghiogheny River. Until within the last three years, the water of the Upper Monongahela River never contained free acid, while now it is quite marked for months at a time. The amount of free acid and sulphate in the Youghiogheny River has also increased very greatly in the last six years, the Youghiogheny always being much worse than the Monongahela River.

In closing the discussion, Mr. Roberts said as follows:

The admirable scheme for discussing acid conditions in our water courses, arranged by our Secretary and Vice-President Handy, elicited from our expert members and invited guests much new and important information. As Mr. Knowles and others stated, the possibilities and cost of projects looking to the abatement of impurities in our streams should be made clear to the public, and this can only be done after a thorough discussion of all the data bearing upon the subject. The range that such discussions are likely to take may be very great, but the Society is to be congratulated for having pointed out at least the direction in which to look for the most beneficial results.

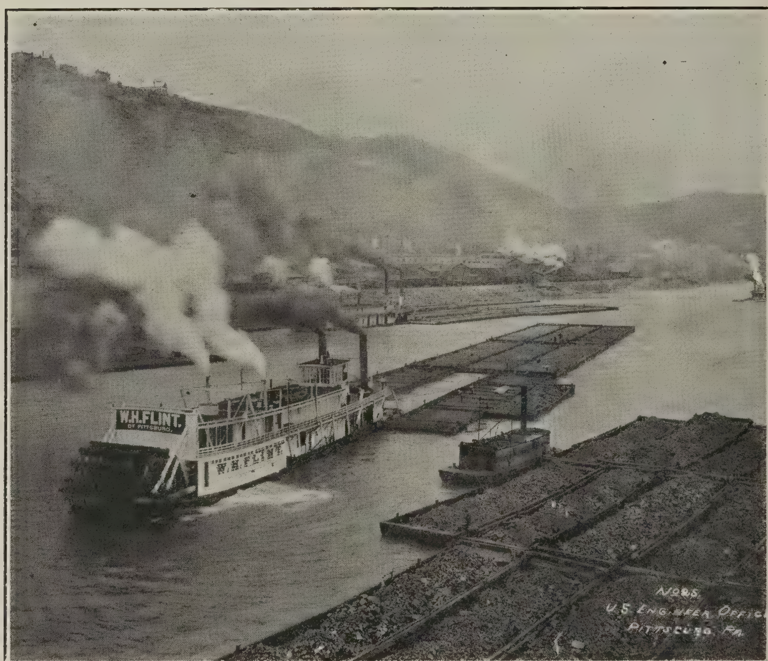


Fig. 6. Departure, from Pittsburgh, of a small coal fleet destined for Louisville.

The writer learned from the discussion that not alone free acids, but, as well also, provision must be made for the elimination of the ferrous sulphate and other salts conveyed by streams issuing from coal mines if our streams are to be restored approximately to their original normal conditions. To take care of such bulky solids in the mine waters means, of course, numerous, and in the case of our largest mines, extensive reduction plants.

One of the speakers estimated that the cost of neutralization at

certain of the mines where sulphur was present in very considerable quantities, would be for reagents, labor, etc., about 30 cents a ton of the total coal mined, which is certainly a statement well calculated to attract serious attention. Other speakers, while fully aware of the cost but with a full realization of the damage to the public welfare to be averted, took the hopeful view that it seemed within the range of possibility to do the work and at the same time make by-products of sufficient value to offset the cost. Pigments are suggested as by-products, but, great as the demand may be, it is easy to conceive the market for pigments might soon be overstocked. Other markets for the output of fluid coal mine wastes will, of course, be sought, and in this connection the writer would

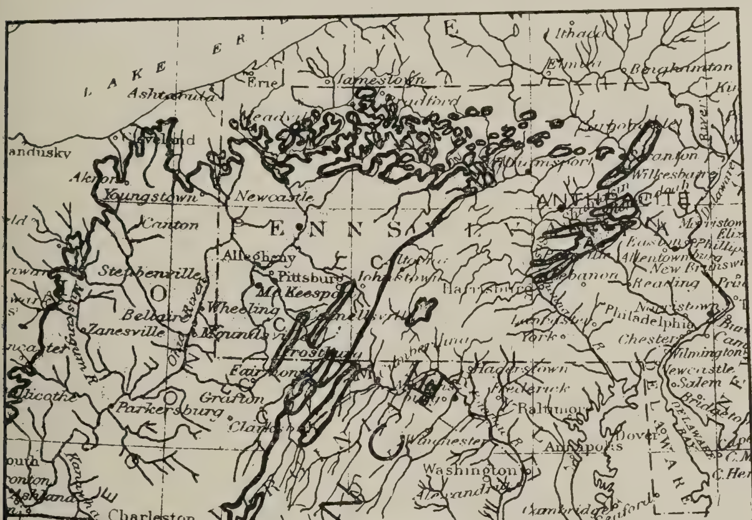


Fig. 7. Map showing location coal fields in relation to drainage in Pennsylvania and the immediate vicinity.

suggest to his friends, who have the requisite technical knowledge, the study of methods of making fertilizers from the mine waters. In former days in the old way of making coal gas, a portion of the waste products containing lime, as one component, had a salable value as a fertilizer, "not much, but still something."

It is well known that there are extensive areas of porous and lean soils in the eastern states where gypsum, or sulphate of lime, would be an excellent fertilizer, specially good as a top dressing for clover, using 200 to 300 pounds per acre. Clover stubble and roots, it is well to know, add more humus to soils than any other of our extensive field crops. This indirect method of securing nitrogen for the land is well worthy of experimental trial. It may be possible to concentrate mine waters by heat to some desired degree, and while still hot allow it to percolate through crushed limestone. If a merchantable gypsum does not result, the writer would advise

the ignition of the residum, until only solids would be left to go in the waste bank.

On the basis of 1 pound of coal evaporating 8 pounds of water, with fuel at 80 cents a ton, the drainage of a mine discharging a half gallon per second and producing 500 tons of coal daily, the cost should not exceed 3½ cents per ton on the output of the mine. Far better this than to permit so much acidulated water to reach the streams. The writer, however, has faith in the idea that our chemists when they turn their attention to it will discover some new source of National wealth from mine wastes.

In the past decades agriculture in America has been for a great part conducted on very wasteful methods. There was too much area for the men to work, and after exhausting the virgin mould from the soil the farmers' sons would "move West," leaving the old men at home. American inventors turned their attention too much to crop harvesting, to the neglect of the soil which produced the crops. It is rapidly becoming a desperate situation east of the Mississippi, the last census showing a diminishing population in a great many counties, with even the State of Iowa on the backward trail.

The millions of tons of fluid waste from the mines, let us continue to believe, will soon no longer be accounted a waste, but a dividend maker for the country.

It is rather ominous to hear several of the speakers state, *first*, that abandoned coal mines are more productive of acids, etc., than are active mines, and, *second*, that the percentage of sulphur in the West Virginia coal was much greater than that in the lower river country. Such facts are quite sufficient to explain the marked increase in acid conditions noticed in the Monongahela River in the past two or three years. We are therefore doomed, it appears, to grow from bad to worse on an ever-increasing ratio.

Mr. Dole stated that whereas in some other coal fields there was not a great population below them, but in Pennsylvania the drainage from every coal mine west of the Allegheny Mountains must necessarily pass through Pittsburgh and through the densely populated surrounding counties. If all the contaminated brooks and creeks were indicated on the map, the "affected area" would be shown to be very great. In some townships potable water for cattle is scarce, unless resort be had to pumping, but pumping and stall feeding will never make cheap beef.

Mr. Snyder stated that not only have the fish disappeared from many of our streams, but that the country is being forsaken by other forms of animal life. We may not fully realize the import of such a statement, but it may have a very serious meaning. There is such a thing in nature, the biologists tell us, as an equilibrium of beasts, birds, fish, insects, preying on each other in the happiest manner imaginable to the great relief of the boss animal, man; but if the series be disturbed by the elimination of certain of the predatory species, other and perhaps very objectionable life forms, free to propagate, may come to afflict us.

Thomas Lincoln Casey

Thomas Lincoln Casey (see frontispiece) was born at Sacketts Harbor, N. Y., May 10, 1831, where his father, Bvt. Maj. Gen. Silas Casey, was then stationed. He entered the United States Military Academy on July 1, 1848, and was graduated and commissioned a Brevet Second Lieutenant in the Corps of Engineers, July 1, 1852. He served at West Point, N. Y., with the company of sappers, miners, and pontoniers during 1852, and was assistant engineer in the construction of Fort Delaware and on the works of river and harbor improvement in Delaware River and Bay until 1854. On June 22, 1854, he was commissioned Second Lieutenant, Corps of Engineers. From 1854 to 1859 he again served at the Military Academy as Assistant Instructor in Practical Military Engineering and as Principal Assistant Professor of Engineering. He was promoted to First Lieutenant, Corps of Engineers, December 1, 1856. From 1859 to 1861 he commanded a detachment of Engineer troops in Washington Territory and was engaged in survey work and road work in Washington and Oregon.

On the outbreak of the Civil War he was assigned as assistant engineer at Fort Monroe, Virginia, and was on the staff of the general commanding the Department of the Virginia from June to August, 1861. He was promoted Captain, Corps of Engineers, August 6, 1861, and was assigned as superintending engineer of the permanent defenses and field fortifications along the coast of Maine, on which duty he was kept until July 25, 1866, except for a short period when he was placed on special duty with the North Atlantic squadron during the first expedition against Fort Fisher, North Carolina, in December, 1864. He was promoted Major, Corps of Engineers, October 2, 1863, and Brevet Lieutenant-Colonel and Brevet Colonel, March 13, 1865, "for faithful and meritorious services during the Rebellion." During his detail at Portland, Maine, he constructed Forts Scammell, Gorges, and Preble in Portland Harbor, Fort Popham at the mouth of the Kennebec, and Fort Knox at the Narrows of the Penobscot. In November, 1867, he was transferred to the Office of the Chief of Engineers and placed in

charge of the Division of Fortifications in that office, where he served for ten years when he was placed in direct charge of the construction and maintenance of the State, War and Navy Building, the Washington Aqueduct, Public Buildings and Grounds, and other public works in the District of Columbia. He was continued in charge of this work until the completion of the State, War and Navy Building, in March, 1888. He was promoted Lieutenant-Colonel September 2, 1874, and Colonel March 12, 1884.

He had hardly settled into the construction of the State, War and Navy Building when, on June 25, 1878, he was appointed Engineer of the Joint Commission charged with completing the Washington Monument. He was mainly responsible for the completion of the Monument in its present form, taking charge of the work when the part of the Monument already erected, 173 feet in height, had to be straightened, as the foundations had settled unevenly; and the original methods used by him are well known to the engineering profession. From 1886 to 1888 he was stationed in New York City as President of the Board of Engineers for Fortifications, retaining charge of the State, War and Navy Building and of the Washington Monument. On July 6, 1888, he was appointed Brigadier-General and Chief of Engineers.

Notwithstanding the many and onerous duties connected with this office, on October 2 of the same year, by act of Congress, he was placed in charge of the construction of the Library of Congress. General Casey was retired from active service May 10, 1895, but was continued in charge of the building of the Library of Congress, which was practically completed when he suddenly died, at Washington, D. C., March 25, 1896, aged 65 years. At the time of his death he was one of the engineers appointed by the Dock Board of New York City to examine and advise upon the construction of its extensive dock system.

General Casey's work in Washington is so well known throughout the world that were he buried here in Washington the epitaph of Sir Christopher Wrenn might well be paraphrased to read, *Si monumenta requiris, circumspice.*

General Casey was a member of the National Academy of Sciences, an officer of the Legion of Honor of France, a member of the Order of the Cincinnati, the Loyal Legion, the Philosophical Society of Washington, Century Association of New York, New England Historic Genealogical Society, and was at one time a Director in the American Society of Civil Engineers. He is buried on the Casey farm, about 7 miles north of Narragansett Pier, R. I.

Church Built, at Petersburg, by Engineers During Civil War*

BY

Capt. W. J. GEORGE

In the fall of 1864 the Army of the Potomac was not ordered into winter quarters, but the Fiftieth New York Engineers, as was always customary with them if they only expected to stay a few days or weeks in a camp, always contrived to make the camp as comfortable as possible for the officers and members of the regiment.

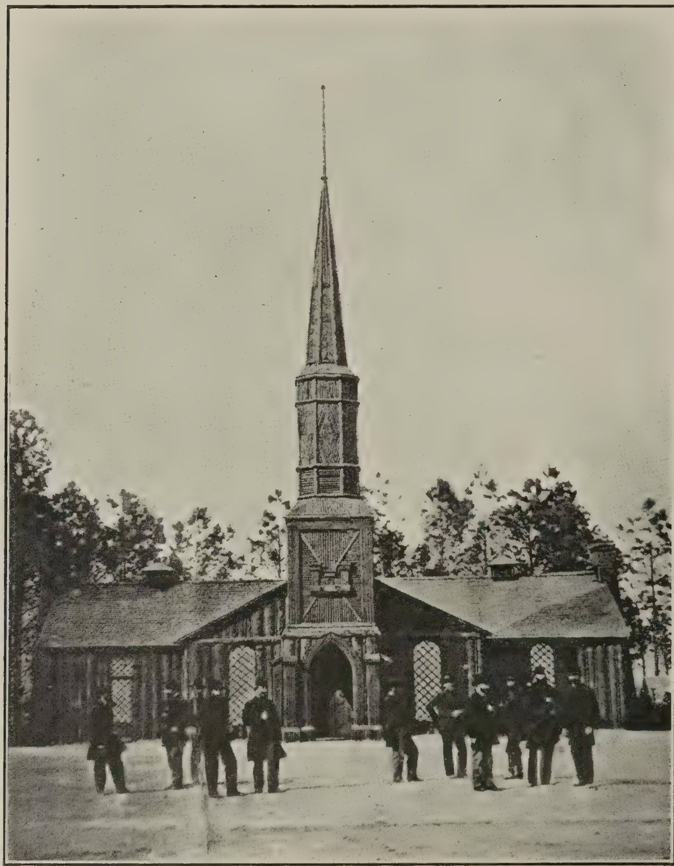
Our camp there was located near Poplar Grove Church, in the rear of Fort Fisher on the siege line at Petersburg. The regiment went into camp in a grove, the large portion of which was original growth of heavy pines. Another part of our camp ground was covered with white oak. The large, commodious parade ground of our camp, lying between the line officers and field officers quarters, was composed of these heavy pines. As soon as we were located, the regiment started in to clear the ground and build quarters, which they did—handsome ones for field officers, Gothic style, giving them quite a homelike appearance by the little extra touches that the mechanics of the regiment placed on them in the way of ornamental decorations, etc. After the quarters were built for the officers and men, they thought it would be very appropriate to build a church, so the logs that were cut out of the parade ground were squared on the three sides, with the bark left on one side, as you will notice in the photograph of same. The shingles were made by hand out of the same timber. Even the floor of the stage (12 by 12) was made with the bark side down and hewn side left up, making a very solid stage indeed. The steeple, as you will notice, was made out of small pines, found in another part of the camp ground, just beginning to grow, only from an inch to an inch and a half thick. Other parts were made out of still larger ones, such as borders of the diamond that you will notice in the base of the spire.

In this church or hall, minstrel performances were given nearly every night during the week while we remained in camp during the years 1864 and 1865. On Sunday it was occupied by the chaplain of the regiment for religious services. There was a marriage took

*Extract from a letter of Captain George to Capt. A. B. Dunning, Co. A, Engineer Battalion, National Guard of Pennsylvania.

place in it during our occupancy of these quarters, one of the staff of Meade's headquarters. Many officers and soldiers of infantry commands within a radius of several miles of the camp attended these entertainments on different occasions, and they were all high in their praise of the beautifully arranged hall or church erected by our regiment.

The corral in the camp was made out of the white oak timber,



Church built by Engineers during the Civil War.

making the animals comfortable during our stay in this camp. As you know, our regiment had charge of all the ponton trains of the Army of the Potomac. They were all centered at this camp during this winter, and the many mules and horses attached to the ponton trains and horses of the officers, all of whom were mounted, field and line, together with their orderlies, made quite a large corral for one regiment.

The Corps of Engineers and the Isthmian Canal*

BY

Lieut. JAMES GORDON STEESE
Corps of Engineers

As is well known, the construction of the Panama Canal was put into the hands of Army Engineers on April 1, 1907, the date of the resignation of Mr. John F. Stevens, Chairman and Chief Engineer of the Isthmian Canal Commission. Engineer officers, however, have been connected with the investigations looking to the construction of an isthmian canal since the early days. The following brief account was compiled from records, reports, etc., on the Isthmus; an exhaustive search of public documents in Washington would undoubtedly disclose additional names.

On the outline map of Central America are indicated the various proposed routes for an inter-oceanic canal or railroad.

The route for the Panama Railroad was laid out by engineers under Col. George W. Hughes, United States Topographical Engineers, in 1849, who found a yet more favorable line than that discovered by Messrs. Stephens and Baldwin, to whom is due the credit for securing this route. A survey for a railroad across the Isthmus of Tehuantepec was made, in 1852, by the Scientific Commission, under the direction of Maj. J. G. Barnard, Corps of Engineers.

The American Atlantic and Pacific Ship Canal Company, a private corporation holding a concession from Nicaragua, had a survey of the Nicaragua Canal Route made in 1850-1852. The report of this survey was submitted by President Fillmore, at the request of the company, to Col. J. J. Abert and Lieut. Col. W. Turnbull, United States Topographical Engineers, for their inspection and opinion. They reported, March 20, 1852, that the

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*For history and description of the Panama Canal, see "A Brief History of the Panama Canal," by Lieut. C. K. Rockwell, PROFESSIONAL MEMOIRS, Vol. I, p. 164, and "The Panama Canal," by Col. H. F. Hodges, ibid., p. 355.

plan was practicable, but recommended some changes and modifications.

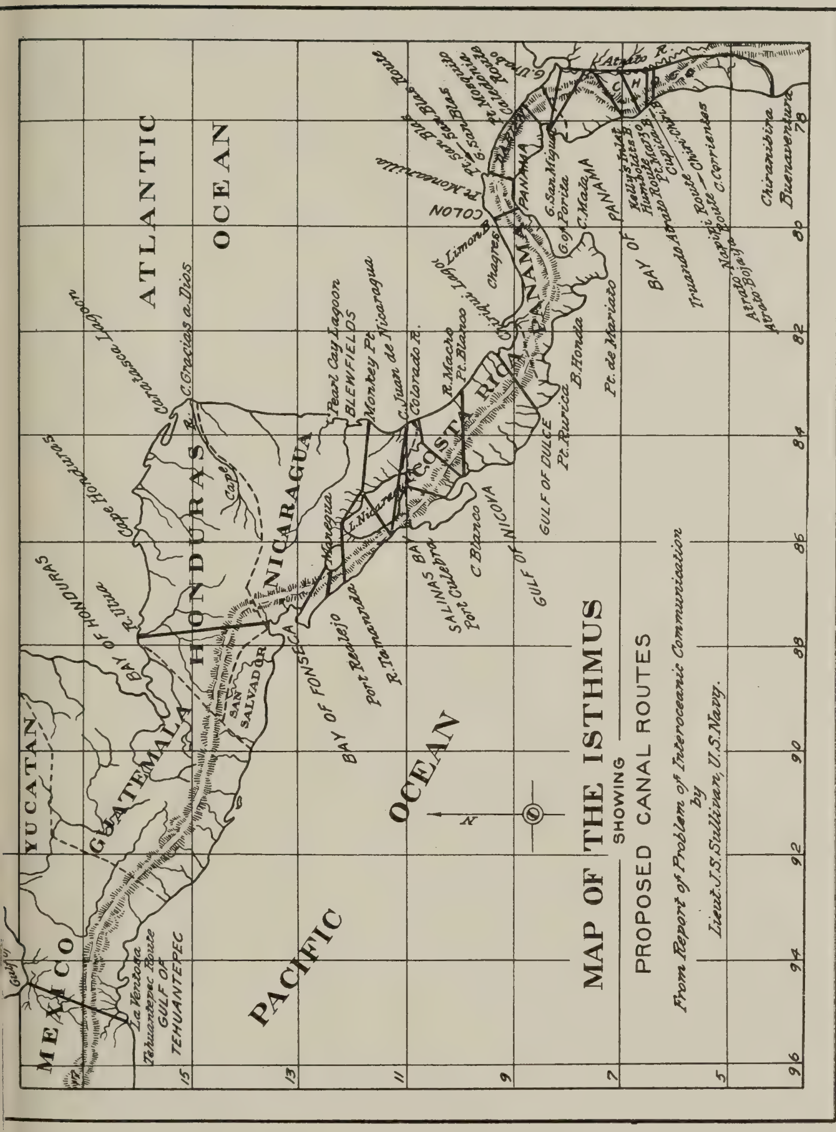
Lieut. Nathaniel Michler, United States Topographical Engineers, verified and reported on the Atrato-Truando Route in 1857-1858. This was the first line in the Atrato Valley that could base its claim of practicability upon full and reliable data, and, moreover, was the first line pronounced feasible for a canal without locks or dams.

Lieut. J. St. C. Morton, United States Topographical Engineers, was one of the four members of a commission to report on the Chiriqui Route in 1860. Gen. A. A. Humphreys, Chief of Engineers, was chairman of a committee of three to investigate the causes of the deterioration of the harbor of Greytown (Nicaragua Route) in 1865.

In 1872, President Grant appointed an Interoceanic Canal Commission of three members, of whom Gen. A. A. Humphreys was chairman. This commission, after considering the available data, and carrving on extensive investigations and surveys of its own over the various routes, from the Isthmus of Tehuantepec to the Atrato River, reported in 1879 in favor of the Nicaragua Route. Among the reports submitted to the commission were those of Maj. Walter McFarland, Corps of Engineers, on the Nicaragua, Darien, and Atrato River routes. Major McFarland was assisted in his surveys by Capt. W. H. Heuer, Corps of Engineers. The other reports were mostly by United States naval officers, many of whom made repeated surveys and reconnaissances during the entire time that the canal project has been under consideration.

The French activities at Panama date from 1876; during the succeeding ten years American interest almost died out. As the French Company became involved in financial difficulties, interest in the Nicaragua Route was revived, culminating in the Nicaragua Canal Board, appointed in 1895, and known as the Ludlow Commission. It consisted of three members, of whom the chairman was Lieut. Col. William Ludlow, Corps of Engineers. Col. Peter C. Hains, Corps of Engineers, was one of the three members of the Nicaragua Canal Commission, appointed in 1899.

In the meantime, Gen. Henry L. Abbot (Colonel, Corps of Engineers), had been invited to become a member of the Comité Technique, appointed by the Board of Directors of the New Panama Canal Company (French), which was organized in October, 1894. The Comité Technique was composed of seven French engi-


MAP OF THE ISTHMUS
SHOWING
PROPOSED CANAL ROUTES
From Report of Problem of Interoceanic Communication
by
Lieut. J.S. Sullivan, U.S. Navy.
ATLANTIC
OCEAN
PACIFIC
OCEAN
MEXICO
YUCATAN
GUATEMALA
HONDURAS
SAN SALVADOR
NICARAGUA
COSTA RICA
PANAMA
BAY OF HONDURAS
BAY OF FONSECA
GULF OF TEHUANTEPEC
Tehuantepec Route
GULF OF NICOYA
GULF OF DULCE
BAY OF PANAMA
COLON
BLEWFIELDS
Chagres
Atrato R.

neers and seven foreign members, and submitted its final report November 16, 1898. General Abbot was then chosen by the board of directors of the New Panama Canal Company to be one of its representatives on the Commission of Five, contemplated by its "Statuts." This commission rendered its final report on February 28, 1899. General Abbot continued to be its consulting engineer from the time of the adjournment of these bodies up to the time of the final transfer of its property to the United States in 1904. He has written several books and numerous papers, published in both English and French, which are authoritative on the canal situation.

We now come down to the present project. Col. Peter C. Hains and Lieut. Col. O. H. Ernst, Corps of Engineers, were two of the nine members of the Isthmian Canal Commission of 1899-1901, which investigated all nineteen canal routes and reported in favor of Nicaragua, in view of the fact that the French Company demanded \$109,000,000 United States gold for its rights. When the French Company agreed to the \$40,000,000 figure, proposed by the Commission, it submitted a supplementary report in favor of Panama, the more desirable route from an engineering point of view.

The "Spooner Act" authorized the purchase of the French rights and the construction of the Panama Canal. In March, 1904, while the negotiations were being closed, Maj. W. M. Black and Lieut. Mark Brooke, Corps of Engineers, were sent to the Isthmus to check up the French work, property, etc.

There were no members of the Corps of Engineers on the Isthmian Canal Commission of 1904, but Capt. C. E. Gillette accompanied the Commission on its first visit to the Isthmus in April, 1904, detailed for the special service required by the Commission relating to a study of conditions as they existed at the time of taking possession of the canal properties by the United States. Later, Major Gillette submitted a project for a lock canal for the consideration of the International Board of Consulting Engineers of 1906.

Major Black returned to the States with the Commission before the transfer took place. When the purchase was completed and the United States was entitled to enter into possession thereof, the Attorney General instructed Lieut. Mark Brooke to take possession of all of the canal properties. This transfer of properties was made on the morning of May 4, 1904, and instructions were

at once given to Lieutenant Brooke by the Isthmian Canal Commission to continue operations with the same force of employes and laborers as were engaged upon the work under the New French Canal Co. at that time. Lieutenant Brooke's declaration (sometimes referred to as the "Memorandum Receipt for $40,000,000") follows:

I, Mark Brooke, officer of the Corps of Engineers of the Army of the United States of North America, declare and state the following:

To-day, the fourth of May, Nineteen hundred and four, early in the morning, in my capacity as representative of the Government of the United States of North America, I came into the building situated in the city of Panama, known in that city by the name of "Hotel de la Compagnie," in which are located the central offices of the New Panama Canal Company, for the purpose of receiving in the name of my principal, the Government of the United States of America, all the properties, personal and real, of the abovenamed company, which are located in the Isthmus of Panama.

After having shown my authority and instructions, the Director of the New Panama Canal Company made formal delivery to me of the said property, personal and real, in the following manner:

He delivered the keys of the buildings and inventories of the properties, called together the principal employes of the service, and in my presence gave them instructions to place at my orders all the material in the storehouses of the company, and the storehouses themselves, and finally, also in my presence, he sent by letter and telegraph the same orders and instructions to all the employes of the company living in Colon and on the line between that city and Panama.

In consequence, I declare in the name of the Government of the United States of North America, which I represent in this act of transfer, that I acknowledge having received all the properties, personal and real, that belonged to the New Panama Canal Company, which have passed into the possession of the Government of the United States of North America, my principal.

This receipt is written and signed in French, English, and Spanish.

(Sgd.) MARK BROOKE,
2nd Lieutenant, Corps of Engineers,
U. S. A.

Panama, *May 4th, 1904.*

Lieutenant Brooke remained in charge of the work until the arrival on the Isthmus of Governor Davis, who was placed by the Commission in charge of the canal construction work. It was carried on with but slight modifications of French methods with

Major Black at the head of the engineering staff, comprising the direction of all engineering works, construction and maintenance of buildings, roads, telegraphs, water supply and sewerage systems, and other auxiliary works, and the prosecution of scientific investigations, until the arrival of the Commission's Chief Engineer, Mr. John F. Wallace, who took charge of the work July 1, 1904.

Gen. Peter C. Hains and Col. O. H. Ernst were two of the seven members of the Isthmian Canal Commission of 1905-1907. Maj. George W. Goethals was a member of the committee of the Board of National Coast Defense which visited the Isthmus, in 1905, and formulated the first fortification project. General Abbot was one of the thirteen members of the International Board of Consulting Engineers, which reported on the type of canal in 1906. Capt. John C. Oakes was secretary of this board. Gen. W. L. Marshall, then Chief of Engineers, was a member of the Joint Army and Navy Board, which visited the Isthmus in 1910, and reported on the fortifications, for which money was subsequently appropriated. Gen. W. H. Bixby, Chief of Engineers, and Col. George W. Goethals are members of the Board as constituted at present.

The present Commission was appointed in 1907, Majors Goethals, Gaillard, and Sibert being three of the seven members. At present four of the seven members of the Isthmian Canal Commission are officers of the Corps of Engineers, Lieut. Col. Hodges having replaced Mr. Jackson Smith (resigned) in September, 1908. The organization is as follows:

Col. Geo. W. Goethals, *Chairman and Chief Engineer.*
Col. H. F. Hodges, *Assistant Chief Engineer, Chief of the First Division.*
Lieut. Col. D. D. Gaillard, *Division Engineer, Central Division.*
Lieut. Col. Wm. L. Sibert, *Division Engineer, Atlantic Division.*

The following officers of the Corps of Engineers are on duty with the Commission:

Maj. Chester Harding, *Assistant Division Engineer, Atlantic Division.*
Maj. J. P. Jervey, *Resident Engineer, Atlantic Division.*
Maj. G. M. Hoffman, *Resident Engineer, Atlantic Division.*
Maj. F. C. Boggs, *General Purchasing Officer, Chief of Washington Office.*
Capt. W. H. Rose, *Electrical Superintendent, Atlantic Division.*
First Lieut. Geo. R. Goethals, *Assistant Engineer, Office of Chief Engineer.*

Maj. E. E. Winslow is in charge of the plans for the fortifications under the Chief of Engineers. Before becoming a member of the

Commission, Lieutenant-Colonel Hodges was general purchasing officer, with Captain Boggs as his assistant. Maj. Edgar Jadwin was on duty with the Commission from July, 1907, to August, 1911, as division engineer of the old Chagres Division and, later, as resident engineer in the Atlantic Division. Capt. H. W. Stickle was on duty with the Commission from December, 1907, to July, 1911, as assistant division engineer of the old Gatun Division and, later, as assistant engineer in the Atlantic Division. Lieutenants Steese, Edgerton, and Goethals were on duty with the Commission from May, 1908, to August, 1909, as transit men and, later, as instrument men on the Panama Railroad engineering corps. Lieutenant Steese was on duty with the Commission from November, 1910, to February, 1912, as assistant engineer, First Division.

Under the scheme of education, introduced by General Marshall in 1909, for junior officers, all Engineer members of the Class of 1909, United States Military Academy, were on duty and instruction on the Isthmus under the orders of the chairman from December, 1909, to May, 1910. Similarly, the Class of 1910 were on the Isthmus from November, 1910, to June, 1911, and the Class of 1911 is at present on the Isthmus.

Federal and State Power over Harbor Lines*

This suit was brought in the Supreme Court of the District of Columbia to set aside certain harbor lines in the harbor of Pittsburgh, Pennsylvania, so far as they encroached upon land owned by the complainant, and to restrain the Secretary of War from causing criminal proceedings to be instituted against the complainant because of the reclamation and occupation of its land outside the prescribed limits. The Court of Appeals of the District affirmed a decree sustaining a demurrer to the bill, and the complainant appeals.

The allegations of the bill, in substance, are as follows:

The complainant, a corporation of the Commonwealth of Pennsylvania, is the owner in fee of ''Brunot's Island,'' formerly Chartier's or Hamilton's Island, in the Ohio River, in Allegheny County, Pennsylvania. In 1858, a statute was enacted in Pennsylvania providing for the appointment of commissioners to ascertain and mark the lines of ordinary high and low water in the Allegheny, Monongahela and Ohio rivers in the vicinity of Pittsburgh. The act recited that the lines of land along the shores of the rivers had not been clearly ascertained, and it was important to all persons interested that their several rights and privileges should be defined. After the commissioners' surveys had been completed and the lines located, opportunity was to be afforded in the court, by which they were appointed, for any needed corrections; and the map or plan finally determined upon was to be recorded. The statute declared that ''the lines so approved shall forever after be deemed, adjudged and taken firm and stable for the purposes aforesaid.'' Proceedings were had accordingly and the high and low water lines along the shore of Brunot's Island were definitely fixed. In consequence the bill asserts that all the land, whether or not under water, inside of the commissioners' lines became the property of the owners of Brunot's Island; and that by virtue of the statute, and the action of the commissioners under it in fixing the high water line as a permanent boundary, the right of the owners of the island to accretions beyond that line was taken away, while at the same time they were no longer subject to loss or diminution of their land by reason of its submer-

*Being a decision of the Supreme Court of the United States in the case of Philadelphia Company v. Henry L. Stimson, Secretary of War, dated March 4, 1912.

gence "through the avulsion of floods or freshets or through gradual erosion."

Subsequent to the establishment, in 1865, of the State commissioners' line, a considerable portion of the shore of the island, "on the so-called back channel, within the said high water mark," was washed away from time to time by heavy floods and freshets, so that a large part of the upland was slightly submerged, but not to an extent sufficient to permit of navigation. Some years ago, the United States Government, in order to increase the depth of water in the harbor of Pittsburgh, caused a dam to be constructed across the Ohio River a short distance below Brunot's Island, known as the Davis Island Dam. And the effect of this dam, says the bill, by the increase of the depth of water in the channel, was to submerge Brunot's Island to a far greater extent and to make the water over the complainant's land navigable "at certain times, and for certain purposes," where it was not navigable before.

In 1895, the Secretary of War, claiming to act under the authority of section 12 of the act of Congress of September 19, 1890, and knowing that the shore of Brunot's Island had been washed away by floods and freshets, established a harbor line which ran across the complainant's land within the line of the State commissioners. It is further alleged that although the submerged land was generally covered by water, "it was not ordinarily navigable water," and "has never constituted, nor does it now constitute a part of the public navigable waters of the United States;" that no authority was conferred by the act of Congress upon the Secretary of War to regulate or interfere with the use of the complainant's land by the establishment of harbor lines upon the same; and that even if the water over this land was in fact part of the public navigable waters of the United States, without being rendered thus navigable by the construction of the dam, still the Secretary of War had no right so to run the harbor line over the land in question as to deprive the complainant of its use and enjoyment. It was the right of the complainant, the bill avers, to repair the damage caused by floods and freshets and to reclaim the submerged portion by filling in or wharfing, "keeping at all times within the lines of the part that had been torn away by the violence of the waters."

In 1907, the Secretary of War, claiming authority under section 11 of the act of Congress of March 3, 1899, against the complainant's protest, changed the harbor line. The report of the United States engineer at Pittsburgh stated that the conditions of high and low water had not changed since 1895, but as along a part of the shore of the island, the harbor line of 1895 ran several hundred feet outside the high-water mark as it then existed, it seemed advisable to change it so as to coincide with the actual high-water mark. A copy of the report with the order of the Secretary of War, dated February 23, 1907, was annexed to the bill and made a part of it. In this it is stated that the location of the proposed

harbor lines was within the bed of the stream as it existed as a physical fact.

The bill further shows that to facilitate the delivery of coal for the operation of its power house on the island, the complainant desired to reclaim a part of it which had been submerged by establishing a coal wharf on the back channel, where both the harbor line of 1895 and that of 1907 "ran some distance landward of the said State commissioners' high water line." According to the proposed plans, the wharf or pier was to extend over the complainant's land and to cross both of the harbor lines to the State commissioners' line. While these plans were being perfected, the Secretary of War, through his representative, the United States engineer officer at Pittsburgh, declared to the complainant that it had no right to build upon its lands across either of the harbor lines, and he refused to permit the complainant to reclaim its land or to build its wharf thereon outside the harbor line of 1907. He threatened that if it undertook to do so, he would prevent it and cause the complainant and its employees "to be prosecuted and fined by the authorities of the Federal Government" for violations of the acts of Congress of September 19, 1890, and March 3, 1899. It was further charged that if the Secretary of War had authority to fix the original harbor line of 1895, that his power was exhausted by what was then done, and that the harbor line of 1907 was wholly unauthorized.

In consequence of the severe penalties prescribed by the acts of Congress for the construction of buildings, piers or wharves outside any harbor line established by the Secretary of War and by reason of the defendant's threats of prosecution in case the complainant carried out its plan of reclamation and the construction of its wharf, the bill avers that the complainant is prevented from making use of its property; that the defendant's action constitutes a taking of its property for public use without just compensation; that it is subjected in its endeavor, so long as the harbor line remains unmodified, to a multiplicity of criminal prosecutions; and that the harbor line is a cloud upon its title.

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Here follows a discussion of the demurrers by the defendant and decisions in regard to purely legal points as to jurisdiction of the court, etc.—Ed.

* * * * * * *

Fourth. Assuming that the court had jurisdiction, we are brought to a consideration of the equity of the bill.

It has been held that the establishment of a general system of harbor lines for the protection of commerce and navigation, is not of itself an injury to property and can not be restrained. *Yesler* v. *Washington Harbor Line Commissioners*, 146 U. S. 646, 656; *Prosser* v. *Northern Pacific R. R. Co.*, 152 U. S. 59, 64, 65. But it has also been recognized that a different question arises when

active measures are taken against an individual proprietor to maintain a location of limits in alleged violation of his private rights and thus to prevent him from enjoying what is asserted to be the lawful use of his property. *Prosser* v. *Northern Pacific R. R. Co., supra.*

The complainant starts with the lines as laid down, in 1865, by the State Commissioners. These lines are averred to be "exactly in accordance with the then existing actual ordinary high and low water marks." The argument is (1) that, independently of the effect of the statute of Pennsylvania, the washing away of the banks, and the submergence of a portion of the island, during the subsequent years worked no loss of title, but that it remained absolute, including the right of reclamation and improvement of the submerged land inside the former line of high water; and (2) that, by virtue of the statute, the boundary was permanently fixed by the State Commissioners' high water line and no subsequent encroachment of the water could affect the rights of the owner.

(1) It is the established rule that a riparian proprietor of land bounded by a stream, the banks of which are changed by the gradual and imperceptible process of accretion or erosion, continues to hold to the stream as his boundary; if his land is increased he is not accountable for the gain, and if it is diminished he has no recourse for the loss. But where a stream suddenly and perceptibly abandons its old channel, the title is not affected and the boundary remains at the former line. *Rex* v. *Yarborough*, 3 B. & C. 91; S. C. 2 Bligh, N. S. 147; *Gifford v. Yarborough,* 5 Bing. 163; *New Orleans* v. *United States,* 10 Peters, 662, 717; *Banks* v. *Ogden,* 2 Wall. 57; *County of St. Clair* v. *Lovingston,* 23 Wall. 46, 67, 68; *Jefferis* v. *East Omaha Land Co.,* 134 U. S. 178, 190-193; *St. Louis* v. *Rutz,* 138 U. S. 226, 245; *Nebraska* v. *Iowa,* 143 U. S. 359; *Shively* v. *Bowlby,* 152 U. S. 1, 35; Hale, *De Jure Maris,* Ch. 1, 4, 6, Hargrave's Law Tracts; *Mulry* v. *Norton,* 100 N. Y. 424. The doctrine that the owner takes the risk of the increase or diminution of his land by the action of the water applies as well to rivers that are strong and swift, to those that overflow their banks, and whether or not dykes and other defenses are necessary to keep the water within its proper limits. It is when the change in the stream is sudden, or violent, and visible, that the title remains the same. It is not enough that the change may be discerned by comparison at two distinct points of time. It must be perceptible when it takes place. "The test as to what is gradual and imperceptible in the sense of the rule is, that though the witness may see from time to time that progress has been made, they could not perceive it while the process was going on." *County of St. Clair* v. *Lovingston, supra.*

We are confined to the allegations of the bill. We have not the advantage of proof and findings, or even of a particularized description in the bill itself, as to the precise character of the alterations in the banks of Brunot's Island which took place during the

long period to which the bill refers. It is alleged "that subsequent to the establishment in 1865 by said Commissioners of the line of high water mark, as aforesaid, a considerable amount of the soil of the shore of said Brunot's Island on the so-called back channel, within the said high water mark was washed away from time to time by heavy floods and freshets, so that a large part of the upland of the island—that is, the land above high water mark, became and was overflowed and slightly submerged by water, but said land was not submerged to an extent sufficient to permit of navigation of any kind thereover." There is no other statement on the point save that the bill asserts that the complainant was entitled to reclaim "keeping at all times within the lines of the part that had been torn away by the violence of the waters."

It is manifest that these allegations are inadequate to support the complainant's contention. The determining words are that the land was "washed away from time to time by heavy floods and freshets," and the reference is to what occurred in many years. This is far from a statement that at any particular time there was such a sudden, violent, and visible change as to justify a departure from the ordinary rule which governs accretion and diminution albeit the stream suffer wide fluctuations in volume, the current be swift, and the banks afford slight resistance to encroachment.

For example, the general principle of accretion, which has that of diminution as its correlative, applies to such rivers as the Mississippi and the Missouri, notwithstanding the extent and rapidity of the changes constantly effected. *Jefferis* v. *East Omaha Land Co., supra; Jones* v. *Soulard,* 24 How. 41; *Saulet* v. *Shepherd,* 4 Wall. 502; *County of St. Clair* v. *Lovingston, supra; St. Louis* v. *Rutz, supra.* In *Nebraska* v. *Iowa, supra,* the question concerned the boundary between the two States, which, by the acts of admission, was the middle of the main channel of the Missouri River. Between 1851 and 1877, in the vicinity of Omaha, there were marked changes in the course of this channel so that in the latter year it occupied a very different bed from that through which it flowed in the former year. The opinion of the court describes in detail the physical conditions along the river. The court said (pp. 368-370): "The current is rapid, far above the average of ordinary rivers; and by reason of the snows in the mountains there are two well-known rises in the volume of its waters, known as the April and June rises. The large volume of water pouring down at the time of these rises, with the rapidity of its current, has great and rapid action upon the loose soil of its banks. * * * The only thing which distinguishes this river from other streams, in the matter of accretion, is in the rapidity of the change caused by the velocity of the current; and this in itself, in the very nature of things, works no change in the principle underlying the rule of law in respect thereto. Our conclusions are that, notwithstanding the rapidity of the changes in the course of the channel, and the washing from the one side and on to the other, the law of ac-

cretion controls on the Missouri River, as elsewhere; and that not only in respect to the rights of individual land owners, but also in respect to the boundary lines between States. The boundary, therefore, between Iowa and Nebraska is a varying line, so far as affected by these changes of diminution and accretion in the mere washing of the waters of the stream.'' And, in the same case, the decision clearly points the distinction between the losses and gains thus described, and an abrupt, visible change where at one place, at a particular time, the river having ''pursued a course in the nature of an ox-bow, suddenly cut through the neck of the bow and made for itself a new channel.'' (p. 370.)

The present case falls within the category first mentioned, and according to general principles of law the owner would bear the losses caused by the washings of the river.

The bill also alleges that ''some years ago the United States Government, in the interest of navigation and in order to increase the depth of water in the harbor of Pittsburgh, caused a dam to be constructed across the Ohio River a short distance below said Brunot's Island known as the Davis Island Dam. The effect of this dam was to very decidedly increase the depth of the water in the channel back of Brunot's Island, and to cause the water of the river to flow higher upon the land of your orator, and to submerge same to a far greater extent and in fact to make said water which submerged your orator's land navigable at certain times, and for certain purposes, which was not navigable before the construction of said dam.''

It will be observed that it is said that the United States caused the erection of the dam in the interest of navigation. The complainant purchased the island subsequently, in the year 1896. And we are not concerned here with the question whether there was any appropriation of land of the former owner by the United States and a cause of action arose to recover its value. *Gibson* v. *United States,* 166 U. S. 269; *United States v. Lynah*, 188 U. S. 445: *Bedford* v. *United States* 192 U. S. 217; *Manigault* v. *Springs* 199 U. S. 473; *C. B. & Q. Ry.* v. *Drainage Commissioners,* 200 U. S. 561, 583, 584. So far as the bill shows the dam was lawfully built, and the allegations with respect to it wholly fail to state any case entitling the complainant to relief by reason of its construction.

(2) The complainant, however, insists that the effect of the Pennsylvania statute was to fix the boundary of the island permanently at the State Commissioners' high water line, and hence that within that line it was entitled to make the desired reclamation and improvement.

This statute (act of 16th April, 1858) provided that the Commissioners' lines approved by the court should ''forever after be deemed, adjudged and taken firm and stable for the purposes aforesaid.'' The Supreme Court of Pennsylvania has held that the purpose of the act was to regulate the rights of the public in respect to navigation and to prevent private rights from being

exercised to the prejudice of the public interest. *Wainwright* v. *McCullough*, 63 Pa. St. 66; *Zug* v. *Commonwealth*, 70 Pa. St. 138, 142; *Poor* v. *McClure,* 77 Pa. St. 214, 219; *Allegheny City* v. *Moorehead,* 80 Pa. St. 118, 139, 140. In *Wainwright* v. *McCullough* (1869), *supra,* that court, holding that the statute was not applicable to disputed boundaries between private owners, considered the navigable character of the rivers to which it related, the extent of riparian rights under the law of the State, and the meaning of the act in the light of the mischief which it was intended to correct. The court said:

"In order to arrive at the legal effect of the lines established by the commissioners under that act, we must ascertain its true purpose; and to reach this, it becomes necessary to examine the navigable character of the rivers Allegheny, Monongahela, and Ohio, and the rights of the riparian proprietors upon their banks. These rivers are among the largest in the State; larger than the Schuylkill and Lehigh, recognized as navigable in the early history of the province, and have been repeatedly held by name to be rivers naturally navigable, and therefore classed with the Delaware and Susquehanna; *Carson* v. *Blazer,* 2 Binney, 478; *Shrunk* v. *Schuylkill Nav. Co.,* 14 S. R. 79, 80; *Hunter* v. *Howard,* 10 Id. 244. Many acts have been passed declaring tributaries of these rivers navigable. But an act perhaps most pertinent to this controversy is that of 8th April, 1785, 2 Sm. Laws, 317, regulating the taking up of lands within the new purchase, of which the 13th section expressly excepts islands in the Ohio, Allegheny and Delaware.

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"This being the navigable character of the stream, the rights of the riparian owners are settled by numerous decisions, a few of which may be referred to: *Carson* v. *Blazer, supra; Shrunk* v. *Schuylkill Nav. Co., supra; Ball* v. *Slack,* 2 Whart. 508, *Zimmerman* v. *Union Canal Co.,* 1 W. & S. 346; *Bailey* v. *Miltenberger,* 7 Casey, 37; *McKeen* v. *Delaware Div. Canal Co.,* 13 Wright, 424; *Tinicum Fishing Co.* v. *Carter,* 11 P. F. Smith, 21, opinion by Sharswood, J., decided last winter at Philadelphia. From these and other cases, it will appear that the absolute title of the riparian proprietor extends to high water mark only, and that between ordinary high and ordinary low water mark, his title to the soil is qualified, it being subject to the public rights of navigation over it, and of improvement of the stream as a highway. He can not occupy to the prejudice of navigation or cause obstructions to be placed upon the shore between these lines, without express authority of the State.

"The case of *Bailey* v. *Miltenberger,* 7 Casey 37, decided in 1856, doubtless had something to do in turning public attention to the shores of the streams surrounding the city of Pittsburg, which led to the passage of the Act of 1858, for the purpose of defining the low and high water lines. It referred to the mistaken idea enter-

tained by some proprietors of making ground for their mills, by depositing cinders on the shore between low and high water marks. 'The Allegheny and many other navigable rivers' (says the opinion) 'do not, at the time of low water, occupy over one-third of their bed; and it would be most disastrous to allow every owner to fill out his land to low water mark.' This state of affairs, for these rivers had been seriously encroached upon at and opposite Pittsburg, no doubt led to the Act of 16th April, 1858, Pamph. L. 326. It begins by a recital, 'Whereas, The lines of lands on and along the shores at the rivers at and near the city of Pittsburg, in the county of Allegheny, have never yet been clearly ascertained, and as it is important to the owners of such lands, the persons navigating the waters of, and the corporations adjacent to, such rivers, and to all parties interested, to know and to have their several rights and privileges in extension and limitation ascertained and defined; therefore,' etc. The first impression arising from this language might seem to be that the law was intended to ascertain and fix these high and low water lines to end all controversies, *private* as well as public. But a careful consideration of its purpose and provisions shows that it is not applicable to disputed boundaries between private owners, but was intended to regulate the respective rights of the public and the land owners, over whose property the right of navigation extends between high and low water lines.

* * * * * * *

"The effect of the lines as established is thus stated: 'the lines so approved shall for ever after be deemed, adjudged and taken, firm and stable for the purposes aforesaid.' If we seek for the 'aforesaid' purposes, the act discloses none but those relating to the public interest and that of the riparian owner. Then if we advert to the power of the state over navigable streams, as stated in the authorities cited, we discover that it is plenary over the subject of navigation and the improvement of these natural channels of commerce, while the ownership of the riparian proprietor is qualified between lines of low and high water. The legislature may, therefore, with great propriety define the bounds of high and low water, by means of a suitable commission, for the purpose of regulating the public right, so as not to conflict with private interests, and to prevent private rights from being exercised to the prejudice of public interests; for example, to prevent the shores from being filled up with great banks of cinders."

In *Allegheny City* v. *Moorehead* (1875), *supra,* the question was presented whether by the fixing of water lines under the act of 1858, title had been vested in the city of Allegheny or lot owners, so as to defeat the claim of the plaintiff Moorehead under a subsequent patent from the State. The court said: "Nor can the operation of the Act of 1858 be expended by the act of the commissioners in running out the low-water line of the northern shore of the river to include a part of what was Killbuck Island. It

was not the purpose of the commissioners to transfer titles, but to mark the boundaries of *riparian* rights, so as to make them certain and permanent in their extent. So it was not the intention of the framers of the Act of 1858 to pass titles to lands, or to ascertain boundaries between individuals; but it was their purpose to regulate the right of navigation along the shores of these rivers by establishing high and low water lines, which would definitely ascertain and fix the extent to which the right could be exercised; and the extent to which the owners of the land could exercise their own rights under the law of the State.''

It is contended for the complainant that the effect of the statute was to secure to riparian owners complete protection against any loss of their land, or of the right to build upon it, by reason of the gradual washing away of the banks of the river; that the State chose to resign to the riparian proprietors its right to such additions from the moving landward of the low water mark, and required the owner at the same time to surrender in the interest of navigation his right to alluvion. In support, the complainant cites the opinion of the Court of Common Pleas No. 2 of Allegheny County in *Briggs* v. *Pheil* (1894), 42 Pittsburgh Legal Journal, p. 18, in which it is said with respect to the same statute: ''At the passage of this act the riparian owner owned absolutely to high water mark, and had a qualified property to low water mark, and outside of the low water mark the title to the soil was in the State. It seems to us there can be no doubt that the State had power to enact that thereafter the legal limits of the property should remain unchanged, either by gradual accretions or by gradual cutting away. This in our opinion was intended to be done and was done by the Act of Assembly and the proceedings thereunder. * * * It seems to us that the establishing of these lines, at least, as between the State and riparian owners, fixing the lines for the future. If the river washes in beyond the high water line the owner may fill up and reclaim the lost land, and on the other hand accretions belong to the State or the municipalities.''

The established doctrine is invoked that the title to the soil under navigable waters within their territorial limits, and the extent of riparian rights, are governed by the laws of the several States, subject to the authority of Congress under the Constitution of the United States. *Martin* v. *Waddell,* 16 Pet. 367; *Pollard* v. *Hagan,* 3 How. 212; *Weber* v. *Harbor Commissioners,* 18 Wall. 57; *Barney* v. *Keokuk,* 94 U. S. 324, 338; *Packer* v. *Bird,* 137 U. S. 661, 669; *St. Louis* v. *Rutz,* 138 U. S. 226, 242; *Hardin* v. *Jordan,* 140 U. S. 371, 382, 402; *Illinois Central R. R. Co.* v. *Illinois,* 146 U. S. 387, 435, 452; *Shively* v. *Bowlby,* 152 U. S. 1, 40-47; *Water Power Co.* v. *Water Commissioners,* 168 U. S. 349, 365. Let it be assumed that the Pennsylvania statute in its regulation of rights, established the commissioners' high water line as the permanent boundary of the island and conferred upon the riparian owner, so far as

it was within the competency of the State to confer it, the right to fill in and to erect structures to the limit of his line, regardless of subsequent changes in the actual high water line caused by the washing away of the banks of the river. What, then, was the power of Congress with respect to the river and what was the extent of the authority conferred upon the Secretary of War?

When the Secretary of War, in 1895, fixed harbor lines he dealt with the stream as it then existed. Whatever right the owner of the island may have had under the State law to reclaim the submerged land within the former line of high water, had not been exercised. The bill, in alleging that the new harbor line ran across the complainant's land, must taken to refer to the submerged land already described. This is the import of its allegations and is shown by the record of the War Department annexed to the bill. In establishing this line, the Secretary of War followed quite closely the actual line of high water as it existed in 1895, except in the back channel of Brunot's Island where it ran several hundred feet outside the then high water mark. The change of the harbor line at this point, in 1907, was for the purpose of making the line coincide with the actual high water mark and in the report of the United States engineer who advised the change it was said that the lines as previously established had "not been filled out to, and the river bed on the Brunot Island side, and in the bend referred to" was in "essentially the same condition" as at the time the harbor lines of 1895 were fixed. He added:

"Pittsburgh suffers annually from floods and in my opinion any material contraction of the channel immediately below the city would result in general injury and would produce conditions detrimental to navigation and to harborage, and it is respectfully recommended that the changes in the established harbor lines shown and described on the map inclosed herewith be made, such changes being necessary in preserving and protecting the harbor of Pittsburg.

"The location of the proposed harbor lines recommended in this communication is within the bed of the stream as it exists as a physical fact."

To this stream, as a highway of commerce, the power of Congress extended; a power which "acknowledges no limitations other than are prescribed in the Constitution." *Gibbons* v. *Ogden*, 9 Wheat. 1, 196. The exercise of this power could not be fettered by any grant made by the State of the soil which formed the bed of the river, or by any authority conferred by the State for the creation of obstructions to its navigation. "Commerce includes navigation. The power to regulate commerce comprehends the control for that purpose, and to the extent necessary, of all the navigable waters of the United States which are accessible from a State other than those in which they lie. For this purpose they are the public property of the nation, and subject to all requisite legislation by Congress. This necessarily includes the power to

keep them open and free from obstructions to their navigation, interposed by the States or otherwise; to remove such obstructions when they exist; and to provide, by such sanctions as they may deem proper, against the occurrence of evil and for the punishment of offenders. For these purposes, Congress possesses all the powers which existed in the States before the adoption of the national Constitution, and which have always existed in the Parliament in England.'' *Gilman* v. *Philadelphia,* 3 Wall. 713, 725.

Nor is the authority of Congress limited to so much of the water of the river as flows over the bed of forty years ago. The alterations produced in the course of years by the action of the water do not restrict the exercise of Federal control in the regulation of commerce. Its bed may vary and its banks may change, but the Federal power remains paramount over the stream, and this control may not be defeated by the action of the State in restricting the public right of navigation within the river's ancient lines. The public right of navigation follows the stream (Rolle's Abr. 390; *Carlisle* v. *Graham,* L. R. 4 Ex. 361, 367, 368) and the authority of Congress goes with it. When the State of Pennsylvania established harbor lines and thus undertook to regulate the rights of navigation, its action, however effective as between the State and the riparian proprietors, was necessarily subject to the paramount power of Congress. The State lines can be conceded no permanent force, as against the will of Congress, without substituting for its constitutional authority the supremacy of the State with respect to navigable waters.

It is for Congress to decide what shall or shall not be deemed in judgment of law an obstruction of navigation. *Pennsylvania* v. *Wheeling and Belmont Bridge Co.,* 18 How. 421. And in its regulation of commerce it may establish harbor lines or limits beyond which deposits shall not be made or structures built in the navigable waters. The principles applicable to this case have been repeatedly stated in recent decisions of this court. *Gibson* v. *United States,* 166 U. S. 269; *Scranton* v. *Wheeler,* 179 U. S. 141; *C. B. & Q. Ry. Co.* v. *Drainage Commissioners,* 200 U. S. 561; *West Chicago R. R. Co.* v. *Chicago,* 201 U. S. 506; *Union Bridge Co.* v. *United States,* 204 U. S. 364; *Monongahela Bridge* v. *United States,* 216 U. S. 177; *Hannibal Bridge Co.* v. *United States,* 221 U. S. 194.

In *Gibson* v. *United States, supra,* the construction of a dyke in the Ohio River under the authority of the Secretary of War had substantially destroyed the landing on and in front of a farm owned by Mrs. Gibson ''by preventing the free egress and ingress to and from said landing'' to ''the main or navigable channel'' of the river. The court said (pp. 271, 272, 275): ''All navigable waters are under the control of the United States for the purpose of regulating and improving navigation, and although the title to the shore and submerged soil is in the various States and individual owners under them, it is always subject to the servitude in respect

of navigation created in favor of Federal Government by Constitution. *South Carolina* v. *Georgia*, 93 U. S. 4; *Shively* v. *Bowlby*, 152 U. S. 1; *Eldridge* v. *Trezevant*, 160 U. S. 452. * * * The Fifth Amendment of the Constitution of the United States provides that private property shall not 'be taken for public use without just compensation.' Here, however, the damage of which Mrs. Gibson complained was not the result of the taking of any part of her property, whether upland or submerged, or a direct invasion thereof, but the incidental consequence of the lawful and proper exercise of a governmental power."

Again, in *Scranton* v. *Wheeler, supra,* the question arose with respect to the riparian owner whose access from his land to navigability was permanently lost by reason of the construction by the United States of a pier resting on submerged lands in front of his upland. The court said in its opinion (p. 163): "The primary use of the waters and the lands under them is for purposes of navigation, and the erection of piers in them to improve navigation for the public is entirely consistent with such use, and infringes no right of the riparian owner. Whatever the nature of the interest of a riparian owner in the submerged lands in front of his upland bordering on a public navigable water, his title is not as full and complete as his title to fast land which has no direct connection with the navigation of such water. It is a qualified title, a bare technical title, not at his absolute disposal, as is his upland, but to be held at all times subordinate to such use of the submerged lands and of the waters flowing over them as may be consistent with or demanded by the public right of navigation."

In *Union Bridge Co.* v. *United States, supra,* the Secretary of War found a bridge to be an unreasonable obstruction to the free navigation of the Allegheny River and required the Bridge Company to make certain changes which it was insisted it could not be compelled to make without compensation. The court, after reviewing the authorities, said (p. 400, 401): "Although the bridge, when erected under the authority of a Pennsylvania charter, may have been a lawful structure, and although it may not have been an unreasonable obstruction to commerce and navigation *as then carried on*, it must be taken, under the cases cited, and upon principle, not only that the company when exerting the power conferred upon it by the State, did so with knowlede of the paramount authority of Congress to regulate commerce among the States, but that it erected the bridge subject to the possibility that Congress might, at some future time, when the public interest demanded, exert its power by appropriate legislation to protect navigation against unreasonable obstructions. Even if the bridge, in its original form, was an unreasonable obstruction to navigation, the mere failure of the United States, at the time, to intervene by its officers or by legislation and prevent its erection, could not create an obligation on the part of the Government to make compensation to the company if, at a subsequent time, and for public reasons,

Congress should forbid the maintenance of bridges that had become unreasonable obstructions to navigation. It is for Congress to determine when it will exert its power to regulate interstate commerce. Its mere silence or inaction when individuals or corporations, under the authority of a State, place unreasonable obstructions in the waterways of the United States, can not have the effect to cast upon the Government an obligation not to exert its constitutional power to regulate interstate commerce except subject to the condition that compensation be made or secured to the individuals or corporation who may be incidentally affected by the exercise of such power. The principle for which the Bridge Company contends would seriously impair the exercise of the beneficent power of the Government to secure the free and unobstructed navigation of the waterways of the United States. We can not give our assent to that principle. In conformity with the adjudged cases, and in order that the constitutional power of Congress may have full operation, we must adjudge that Congress has power to protect navigation on all waterways of the United States against unreasonable obstructions, even those created under the sanction of a State, and that an order to so alter a bridge over a waterway of the United States that it will cease to be an unreasonable obstruction to navigation will not amount to a taking of private property for public use for which compensation need be made.''

It must be concluded, therefore, that it was competent for Congress to provide for the establishment of the harbor lines in question for the protection of the harbor of Pittsburgh. It acted within its constitutional power in authorizing the Secretary of War to fix the lines. *Union Bridge Co.* v. *United States,supra* (pp. 385-388); *Monongahela Bridge* v. *United States, supra* (p. 192). That officer did not exhaust his authority in laying the lines first established in 1895, but was entitled to change them, as he did change them in 1907, in order more fully to preserve the river from obstruction. And, in none of the acts complained of did he exceed the power which had been conferred.

The bill failed to show any ground upon which the complainant was entitled to relief and it was properly dismissed.

Decree affirmed.

Selected Articles of Engineering Interest

Compiled by Henry E. Haferkorn, Librarian, Engineer School.

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering-Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40 Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution
(52) Barge Canal Bulletin.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.

BANK PROTECTION.

Concrete mattress bank protection. B. Okazaki. (14), May 16, 1912. D. I.

BARGES.

Comparative costs of repairs to barges of treated and untreated timbers; cost of timber and steel barges compared. (12), April 24, 1912. D.—Comparative life of steel barges and creosoted timber barges. (12), April 24, 1912.—Method of measuring the displacement of material in scow barges. (12), May 1, 1912. D.—Steel barges for Mississippi River improvement work. (12), April 24, 1912. D.—Cost, longevity, and repairs of tow-boats and other pieces of floating plant. G. W. Durham. (30), July-August, 1912. D. I.

BLASTING.

Current practice in blasting and dredging rock under water. (12), April 24, 1912.

BREAKWATERS.

Danish reinforced concrete breakwater. (15), May 4, 1912.—Method of constructing a breakwater at Naples, Italy, using concrete hollow blocks made of puzzuolana Portland cement. (12), June 12, 1912.

CAISSONS. (See also, Cofferdams.)

Cost of concrete caisson dock foundations. (12), May 8.—Large pneumatic foundation of the New York telephone building. (15), June 1, 1912. I.—North main pier of the Quebec bridge. (10), April 12, 1912. D. I.

CANALS.

Beginning operations on Calumet sag canal. (39), May 1, 1912. D. I.—Bow Valley irrigation works of the Canadian Pacific Railway Co. in Alberta. (12), May 1, 1912. D. I.—Economic canal location in uniform countries. L. E. Bishop. (29), v. 74, Dec., 1911. D.—Irondequoit Creek concrete trough. H. J. Knoppel. (15), May 18, 1912. D. I.—Note sur l'utilite d'une concavite du plafond d'un canal. M. Galliot. (1), March-April, 1912. D.

CEMENT.

Autoclave boiling test for cement. H. J. Force. (14), June 13, 1912. I.—Boiling and steam tests of cement. W. P. Gano. (14), May 23, 1912. I.—Contitution of Portland cement. (7), April, 1912. I.—Government adopts new specifications. (40), June, 1912.—How Portland cement should be stored by dealers and users. (34), May, 1912. D. I.—Present status of iron ore cements. (7), April, 1912; (15), April 27, 1912.—Retrogression in the tensile strength of cement. J. M. O'Hara. (29), v. 74, Dec., 1911. D.—Some of the properties of oil-mixed Portland cement and concrete. L. W. Page. (29), v. 74, Dec., 1911. D. I.—Study of sand for use in cement, mortar, and concrete. E. S. Larned. (45), April, 1912.—Tensile tests of cement briquettes exposed to fresh water, river water, and artificial sea water. C. W. Staniford. (14), May 9, 1912. D.—Testing sand for use in concrete and cement mortar. C. M. Chapman. (15), April 27, 1912.

COAST DEFENSE.

Coast defense. Precis of an article from Streffleur's militarische Zeitschrift. H. W. Roberts. (31), May, 1912.

COATINGS.

Methods of testing coatings for cement surfaces. C. M. Chapman. (15), June 1, 1912.

COFFERDAFS. (See also, Caissons.)

A coffer-dam or caisson without timber or iron in its construction. R. L. Harris. (29), v. 24, 1891. 3 plates.—Hydro-electric project of the Mississippi River Power Co.

at Keokuk, Iowa. D. Taylor. (12), May 29, 1912. D. I.—Protecting a coffer-dam from ice and floods. (39), May 15, 1912. I.

CONCRETE. (See also, Panama Canal.)

Anchoring the face slabs of the Belle Fourche dam. (15), May 18, 1912.—Beam and slab construction for facing reservoir slopes. (15), May 18, 1912. D. I.—Concrete mattress bank protection. B. Okazaki. (14), May 16, 1912. D. I.—Concrete vs. stone masonry sea walls. G. L. Bilderbeck. (14), April 18, 1912. D.—Constructing a concrete pile foundation. (14), May 2, 1912. D.—Cylindrical reinforced concrete foundation. (15), April 27, 1912.—Danish reinforced concrete breakwater. (15), May 4, 1912.—A 1,300,000-gallon concrete reservoir. E. W. Robinson. (15), May 11, 1912. D.—Government tests on waterproofing and damp-proofing of concrete and waterproofing compounds. (14), May 2, 1912.—Irondequoit Creek concrete trough. H. J. Knoppel. (15), May 11, 1912. D. I.—Ligno-concrete. (11), April 5, 1912.—Measurement of actual stresses in reinforced concrete structure. W. K. Hatt. (14), April 18, 1912. D. I.—Mechanical appliances on the Panama Canal. J. F. Springer. (5), May, 1912. I.—Metal diaphragm for making water-tight joints in concrete. (12), May 8, 1912. D.—Method of facing and capping with concrete a crib and stone dam. (12), May 29, 1912. D.—Methods of patching and repairing plain and reinforced concrete. (14), April 25, 1912.—New type of steel and concrete construction. (15), April 27, 1912. D.—Note au sujet des blocs artificiels en beton employes au port de Tenes. M. Raby. (1), March-April, 1912.—Ocean pier of reinforced concrete. (15), May 11, 1912. I.—Permeability tests of mortars and concretes of different proportions. (12), April 24, 1912. D.—Placing concrete under a river tunnel through deep water. (15), April 27, 1912.—Placing concrete under water at Louisville. G. D. Crain, jr. (34), May, 1912. I.—Proposed British standard tests for concrete and reinforced concrete. (15), May 18, 1912.—Reinforced concrete in hydraulic works. J. S. Sewell. (14), May 30, 1912.—Reinforced concrete substructure of the Havana docks company's piers. (15), April 27, 1912. D. I.—Some of the properties of oil-mixed cement and concrete. L. W. Page. (29), v. 74, Dec., 1911. D. I.—Tensile tests of cement briquettes exposed to fresh water, river water, and artificial sea water. C. W. Staniford. (14), May 9, 1912. D.—Testing sand for use in concrete and cement mortar. C. M. Chapman. (15), April 27, 1912.—Two examples of reinforced concrete canal lock construction in Russia. (12), June 5, 1912. D.—United States Government tests on reinforced concrete beams. (14), May 9, 1912.—Use in Italy of Pozzuolana with Portland cement for marine concrete works. (12), May 29, 1912.—Useful series of tests on the permeability of concrete. (12), April 24, 1912.

CRANES, HOISTS, ETC.

Hydraulic power. J. Horner. (5), May, 1912. I.—Appareils de levage et manutention. (48), May 1, 1912. D. I.—New type of English dock crane. (14), May 2, 1912. D.—Nouvelles grues de construction. (48), April 15, 1912. D.

DERRICKS.

Construction and erection of a built-up mast for a wireless telegraph station at Melbourne. W. A. Coxen. (Commonwealth Military Journal), March, 1912. I.

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Editorial Notes

Civilian Appointments to the Corps of Engineers

The second examination of civilians for probational appointment as second lieutenants in the Corps of Engineers will take place in September of this year. There are at present several vacancies in the grade of second lieutenant, a number of which may be filled by appointments from civil life. In order to be eligible to take the examination, the applicant must be a graduate of an approved school of engineering, and eligible for the position of Junior Engineer in the Engineer Bureau of the War Department. He must also be between the ages of 21 and 29 and unmarried, and must be authorized to take the examination by the Adjutant-General of the Army. The application made to the Adjutant-General must include a signed statement showing the applicant's date of birth, whether he is married or single, and whether or not he is a citizen of the United States by birth or naturalization. He should accompany his application by the original, or a certified copy, of a diploma, or other sufficient certificate showing graduation in an engineering course of a well-established high-grade technical school, and evidence that he is eligible for appointment as a Junior Engineer in the Engineer Bureau of the War Department.

The examination will be written, and held at various convenient Army posts throughout the United States and in the Philippines. All applicants will take the same examination, and the papers will be marked by one board.

Those candidates who pass a satisfactory examination will be appointed as probational second lieutenants in the Corps of Engineers, on full pay, for one year, and, at the end of that time, will take a final examination in subjects pertaining particularly to the military, and which they will have had an opportunity to study

during the period of their probation. In the first examination for the appointment, the subjects are:

General History.
Elementary French, German, or Spanish (choice by the candidate).
Physics (including Electricity, Magnetism, Sound, Heat, and Light).
Chemistry.
Geology and Mineralogy.
Elements of Electrical Engineering.
General and Practical Astronomy.
Surveying, including Hydrographic and Geodetic.
Descriptive Geometry and Drawing.
Theoretical and Applied Mechanics.
Theory and Practice of Engineering Construction.
Materials of Construction.

The order providing for the examination states, That for a candidate to pass satisfactorily, he must make a general average of at least 80 per centum on the entire examination, and not fall below 70 per centum in any one subject. It is to be hoped that there will be a large number of candidates for this examination.—A. A. F.

Is the Bed of the Mississippi Rising, Due to Levee Construction?

In connection with the subject of levees, it is commonly asserted that their utility is only temporary, inasmuch as the bed of the river rises as the levees are built and will continue to rise, thereby necessitating a continual increase in the height and cross-section of the levees. This is so fallacious to a person who has made a study of the matter that only its apparent acceptance by many intelligent laymen warrants its denial. It probably arises from the fact that the high waters have increased in height as the levees have been constructed, and the people have assumed that the bottom of the river must be rising to account for this increase in height, instead of considering the new conditions that the levees have imposed as compared with the old ones before levees were constructed.

Twenty-five years ago the Mississippi at flood stage was sometimes from 40 to 60 miles wide, and the level of high water was

much lower than it is to-day at any given point in the bottom. No levees then existing were able to withstand the floods, but were overtopped at periods varying from three to ten years. As levee construction has progressed and the levees have become higher and almost continuous throughout the bottoms, the river at high water has been decreased in width to from 2 to 4 miles, with the natural and inevitable elevation of the water's surface in times of flood. But there is not the slightest evidence to warrant the belief that the bottom of the river is rising to a measurable degree; while, on the contrary, every principle of the flow of water tends to show that such is not the case. It is rising throughout very slowly and might be great enough to measure in a thousand years, but that there is any rapid general rise or any local rise in any isolated stretch has never been observed nor is it consonant with accepted hydraulic principles.

At no point in the lower river is there any sudden change in material, either as to character or size of particles, the material throughout being practically uniform and equally erodible. If at any particular locality the river bed were rising the slope below that section would be increased with a consequent increase in velocity, and the bed would be eroded until the slope readjusted itself to former conditions. If there were any particular section of the river where there were great differences in the current velocity there might be some question of the above statement, but within any given length of 100 miles the current is practically the same, and the one argument given above is sufficient to prove that there can be no local rising of the bed.

As far as the river bed is rising at all, it must rise uniformly over its entire length or else the conditions above stated would certainly come into action; but as the bed throughout its entire length rises there is no point where its measure is shown so absolutely as at its mouth, where its advance into the Gulf is about 260 feet a year. On the Lower Mississippi River there are two controlling points: the chain of rocks just above Cairo being one, and the bar at the mouth of the Mississippi, 1,100 miles below, the other. The chain of rocks is fixed; the bar at the mouth of the Mississippi is movable, and the difference in slope produced by this annual increase in

length of 5 miles per century is a measure of the rise in the bed of the river. That the river bed is slowly rising there is no question whatsoever, for the formation of the Lower Mississippi Valley is almost conclusive proof that at one time the mouth of the Mississippi was in the neighborhood of Cairo and that during past ages this mouth has advanced, due to the building up of the delta by erosion from the Mississippi Valley above Cairo to its present position about 1,100 miles below. How many thousand years this has required no one can state, but the rate of progress of the land formation at the mouth indicates that its effect in raising the river bed can not be measured in an hundred years. This building up of the mouth and bed is quite independent of levee construction and its influence upon such construction may be neglected.—W. D. C.

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BRIG. GEN. WILLIAM PRICE CRAIGHILL

CHIEF OF ENGINEERS, UNITED STATES ARMY

1895-1897

BORN 1833—DIED 1909

Description and Cost of Concrete Superstructures for Breakwaters at Harbor Beach, Mich.

BY

Mr. E. J. Duffies
Assistant Engineer; Member American Society Civil Engineers

In 1904 the Government began rebuilding the superstructure of the main breakwater at Harbor Beach, Mich., in concrete. This breakwater, with two other piers, called the north and south piers, form an artificial harbor of refuge on the west shore of Lake Huron, 60 miles north of Port Huron. For several years the Government had been working on plans for rebuilding this breakwater, the superstructure of which had become very weak from age and decay. The breakwaters at Harbor Beach were begun in 1874 and completed in 1886. The main breakwater, which is 4,695 feet long, is built of cribs 38 feet wide and 65 feet long, filled with stone and sunk on the bottom of the lake, which, at this point, is soft rock or hard blue clay containing small stone and boulders. The superstructure extended to an elevation of about 8 feet above the surface of the lake. On the lake face a sea wall, whose cross section in a vertical plane was a right-angled triangle, extended about 7 feet above the deck of the superstructure proper. The lake face of the breakwater was sheeted with 12 by 12 inch vertical timbers 12 feet long, extending down from the bottom of the sea wall. This vertical sheeting was thoroughly drift-bolted to the breakwater and, near the water line, was covered with iron plates 8 by 4 feet by 3/8 inch.

TEARING UP THE OLD SUPERSTRUCTURE.

In removing the superstructure, all the timbers were torn out to a fixed elevation, or as near to this elevation as the position of the cribs would permit. When cribs were level, it was not necessary to tear up the walls of the cribs more than 1 foot below the fixed grade or elevation, which, by the way, was from 3 to 4 feet below the surface. But, if the cribs were tipped and listed, as

they were in a majority of cases, it was necessary to tear up much deeper at one end of the crib. To perform this work a 20-ton steam derrick, placed on a scow, was used. The timber above water was mostly hemlock which, in late years, it was necessary to use in repairs and was badly rotted, so that it was easily removed. From about 1 foot above the average lake level in the summer on the harbor side and 2 feet or more on the lake side, the timber was perfectly sound, there being many places where the timbers showed the pencil lines made during the framing of the original cribs. Such sound timber was torn up with great difficulty. Again, the layers of timber would not always separate at the desired points. This was particularly so below water and frequently four or five timbers would hold together very tenaciously, and would separate only at a point several feet below the desired grade, in spite of the utmost care to separate them at the required grade. This, of course, added to the work, and very laborious work, in releveling the cribs for the concrete footing blocks. In removing the superstructure it was necessary to tear up about 135,000 feet board measure timber per 100 feet of breakwater and to remove about 2,100 cubic yards of stone for the same distance, so as to permit releveling the walls of the cribs at proper grade. An orange-peel bucket was used for removing the stone below water. The two inside, or harbor side, pockets were removed first, leaving the outer, or lake side, pocket to serve as a breakwater against seas while emptying the two inside pockets. The outside pockets were removed during calm weather. What was necessary of the good timber was used in releveling the walls of the cribs, and the stone that was not used in refilling the cribs and the pockets between the concrete footing blocks was used as riprap along the breakwater.

LEVELING-OFF CRIBS; TIMBER.

As mentioned above, the cribs were rarely level, consequently, in leveling off, the timbers at one end of the cribs at least had to be placed 3 and 4 feet below the grade desired and drift-bolted to the walls of the cribs. Each timber must be separately fitted and, as all measurements had to be taken below water, the work was tedious and required great care. To place timbers at or near grade the men had to work in water from 3 to 4 feet deep. The average depth was not sufficient for the employment of divers, so the work was performed by men in wading boots which came

up to their shoulders. Under most favorable circumstances, a gang of seven men could lay from 3,000 to 4,000 feet board measure per day. The timber relaid under water, however carefully done, could not be placed as securely as the original walls of the

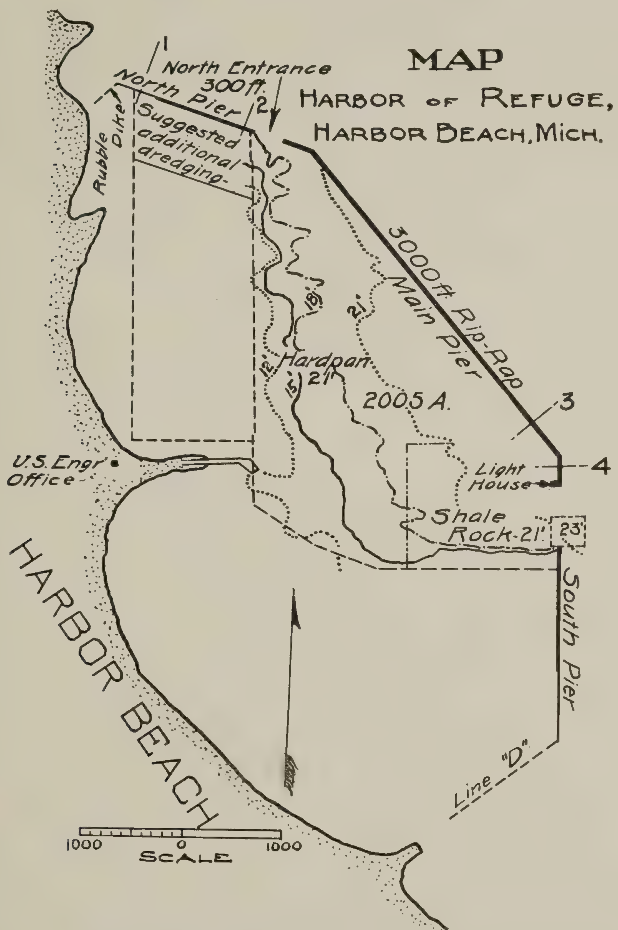

Fig. 1. Plan of harbor of refuge at Harbor Beach, Michigan.

cribs. After the walls were brought to proper grade, the decking was placed for the support of the concrete footing blocks.

STONE.

After the decking was in place on the releveled cribs, selected stone taken from the superstructure was dumped into the spaces

between the pieces of decking. The stone was worked under and around the decking by men, in wading boots, who manipulated the stone with their feet. In order to do this work in 3 or 4 feet of water the men had to work in pairs or three or fours, otherwise they could not keep their feet if there was a slight swell. After the stone was thoroughly worked around the decking and to within about 2 inches of the top, crushed rock or coarse gravel was placed between the planking and leveled off even with the surface of the decking. This work had to be done with great care by the men, as a pebble left on a plank or projecting above the level of the decking would hold up a concrete footing block so that it would not have a uniform foundation on the decking. The operation of placing the crushed rock or coarse gravel was performed at the time of setting the concrete footing blocks.

CONCRETE FOOTING BLOCKS.

The concrete footing blocks for the outside rows and one interior longitudinal row were made 10 feet long, 4.5 feet wide, and 4 feet high. The cross blocks were 4 feet wide, 4 feet high, and 7.5 and 11.5 feet long. The footing blocks had joggles or panels molded in their adjacent vertical faces and upper face for bonding them to each other and to the mass concrete. The blocks were molded on the north pier, by permission of the Engineer officer in charge, and allowed to season at least two weeks before removal. When old enough and sufficient cribs had been leveled to receive them, they were placed on the breakwater. These footing blocks were set by the 20-ton steam derrick used for tearing up. It was found that a little swell or sea interfered very much with the setting of footing blocks. A crew of 12 men could set 20 blocks under favorable circumstances in one day. As stated above, these men leveled off the stone filling at time of setting the footing blocks. After the footing blocks were set, the pockets between the rows and cross blocks were filled with stone to the required height of 2 feet above their upper surface. The two inside rows, or those on the harbor side, had to be chained together until covered with mass concrete to prevent their being washed off the cribs, during storms, into the harbor. At first it was thought if the vertical joggles between the blocks were filled with concrete it would prevent the blocks being moved by the waves, but this method was a failure and only added to the work of realigning or resetting the blocks after a severe storm. The joggles were filled at the time of

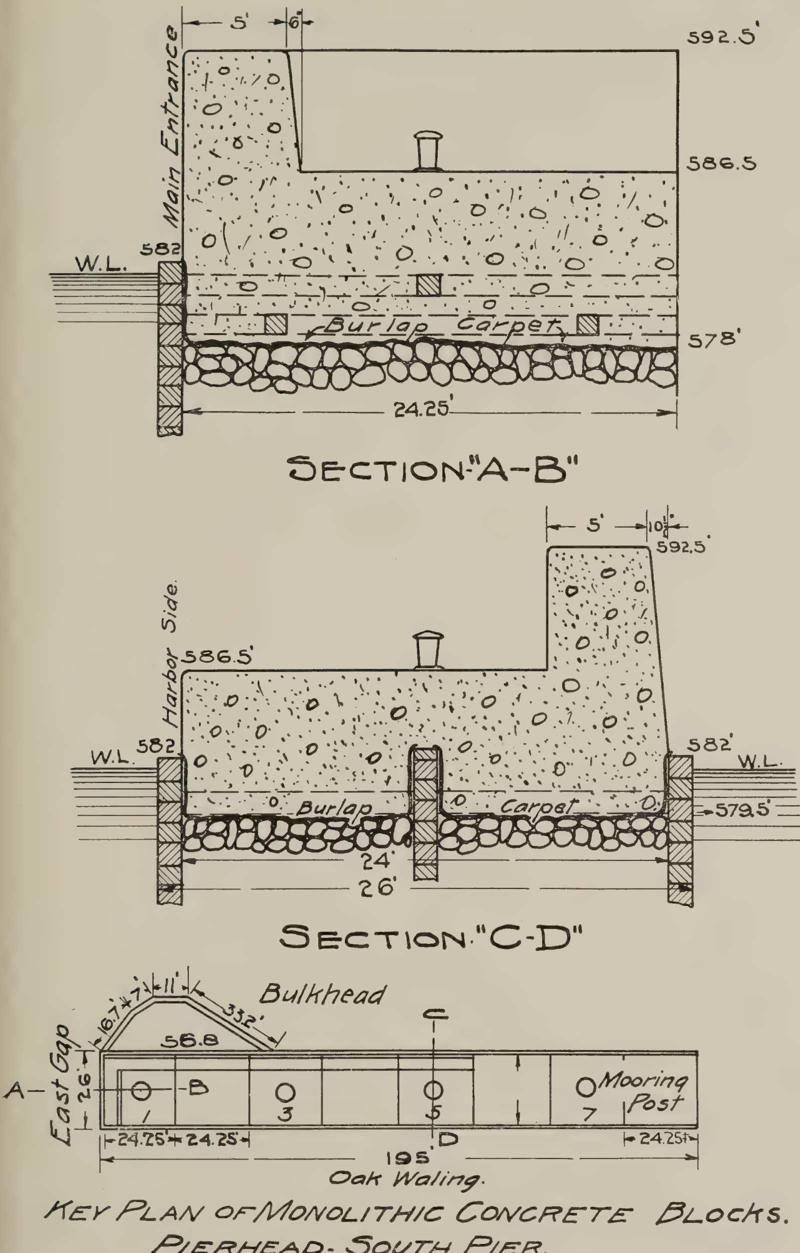

Fig. 2. Plan and enlarged cross section of concrete superstructure for the pierhead, South Pier, Harbor Beach, Michigan.

building the mass concrete, as this method tied the mass concrete and footing block together. Even the outside row of blocks, which were set endwise to the lake face, were moved in several feet during storms.

MASS CONCRETE.

On account of the exposed position of the breakwater, the forms were never put in place until just before beginning the work of filling them. The forms were assembled at a place inside the harbor where they were safe from storms. If, in the morning, the conditions of the sea and weather promised a good day for building a mass block, the form was lifted as a whole from the place where it was set up, placed on a scow and transported to the breakwater, where it was set on the footing blocks and leveled up. The 20-ton steam derrick was used for this purpose. A form could be set up in place ready for filling in less than one-half hour. The work of filling was then begun and the form filled in the usual way from a mixing plant placed on a scow. After the work was well organized, a mass block of nearly 200 yards could be molded in about five and one-half hours on an average.

There were 185 blocks on the main breakwater and it took 185 days to mold these blocks. The greatest number molded in one season was 52, and the greatest number molded in one month was 16. The most of the time was employed tearing up the superstructure and leveling the cribs below water and setting the footing blocks.

Leveling the cribs and setting the footing blocks was slow and tedious work. As a comparison of the relative time required to prepare the foundation for the mass concrete, it may be stated that it took 600 working days to tear up the superstructure; 231 days to level off same; and 200 days to set the footing blocks. It took 319 days to mold the footing blocks and 185 to build the mass concrete. The removal of the stone filling was done almost wholly by one orange-peel bucket working practically all the time. On account of leveling off the cribs so far below water, it was necessary to take out more stone than the plans called for, in order that no stone would be in the way when the cribs were releveled. Again, in tearing up the walls the layers of timber would separate a few inches below the point at which they would finally tear apart and below the desired grade. If the stone was not removed lower than this point, a small rock would invariably roll into the crevice and

prevent driving the timbers together again. It can be seen readily how unsatisfactory and laborious was this class of work. When the lake was muddy it was impossible to see anything below the

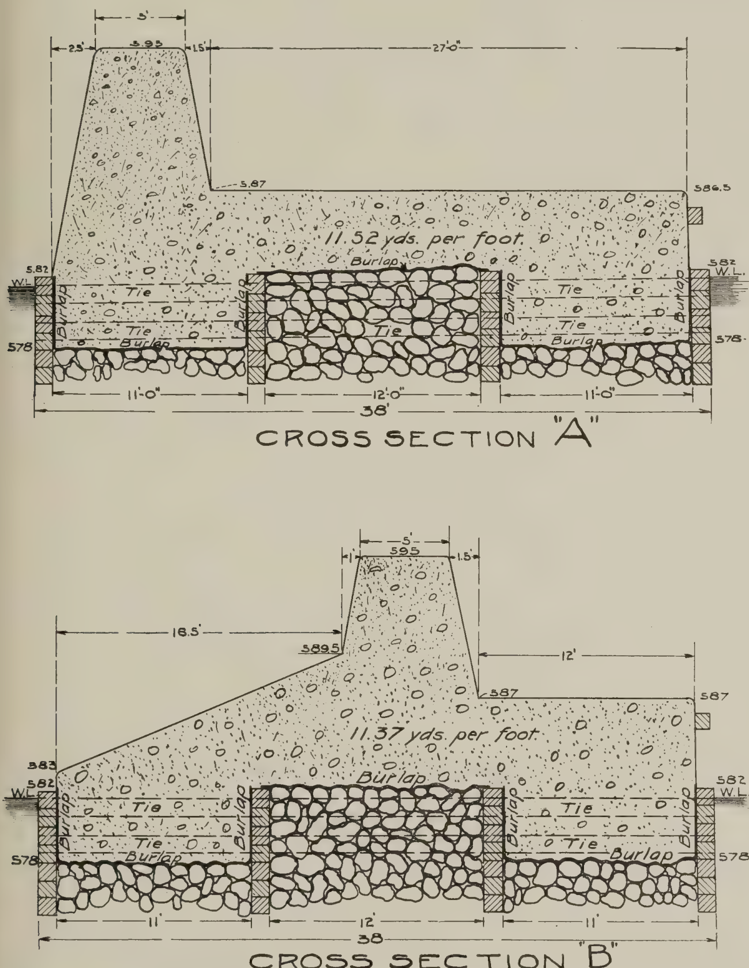

Fig. 3. Proposed cross section for concrete superstructure of the main breakwater, showing how the concrete might have been placed had footing blocks been dispensed with.

surface, except with a water telescope, and anyone who has used that instrument knows its limited field of vision. Then, if there was the least swell or choppy sea, it was impossible to do good

work below the surface. However conscientious a contractor may be, after making several attempts to secure a timber in place and he finally succeeds, he is more than human if he will go further and try to find out if it is down in the best manner. Nor can an inspector find all of the weak points in a piece of work that is hidden from him by a veil of muddy water. Then, after the cribs were finally leveled off to the best of human ability, when the footing blocks came to be set on this carefully prepared foundation, it was found that they did not have a uniform foundation. They would be held up on one corner or in the middle and would rock. While the blocks were made in forms exactly alike, when set on the cribs one would be higher or lower than its neighbor, or would lean away from or toward the adjacent block. After the mass concrete was placed, the additional weight would cause the footing blocks to settle and, on account of the uneven foundation, to crack. This settlement of the footing blocks would also cause unequal stresses in the mass blocks, which resulted in flaking off of corners and edges to some extent. Another defect which manifested itself after a few years was the undermining of the footing blocks. As already stated, the walls of the cribs were not releveled, so that they were as strong as before being torn up. The waves in severe storms would loosen and split the hemlock timber used in releveling immediately under the footing blocks. The receding waves would draw out these pieces of timber by suction and, finally, the stone under the footing blocks. The incoming waves breaking on top of and against the superstructure would cause the footing blocks to settle and slide out. The lake side of the mass concrete would then settle, causing them to crack just inside the parapet. Fortunately, this defect has appeared on one portion of the pier only, but this is an inherent weakness. The walls, before being torn up, had withstood the attacks of the waves with no sign of failure. If all the good timber of the cribs which is torn up below water could be left in place, the cribs would be stronger and a big item of expense would be saved. This part of a breakwater is the vulnerable point, and if the walls could be backed up with concrete at this point, in addition to being undisturbed by rebuilding, they would be additionally strong. If the labor of making and setting footing blocks could be dispensed with another large item of cost would be eliminated.

These considerations lead to the designing of plans for rebuilding of breakwaters by dispensing with footing blocks and of level-

ing only the outside walls of the cribs to a point just above water surface, as the timber in the walls is always found to be sound at this point. To strengthen the walls and to insure the integrity of the cribs should the timber be worn away by the ice or other

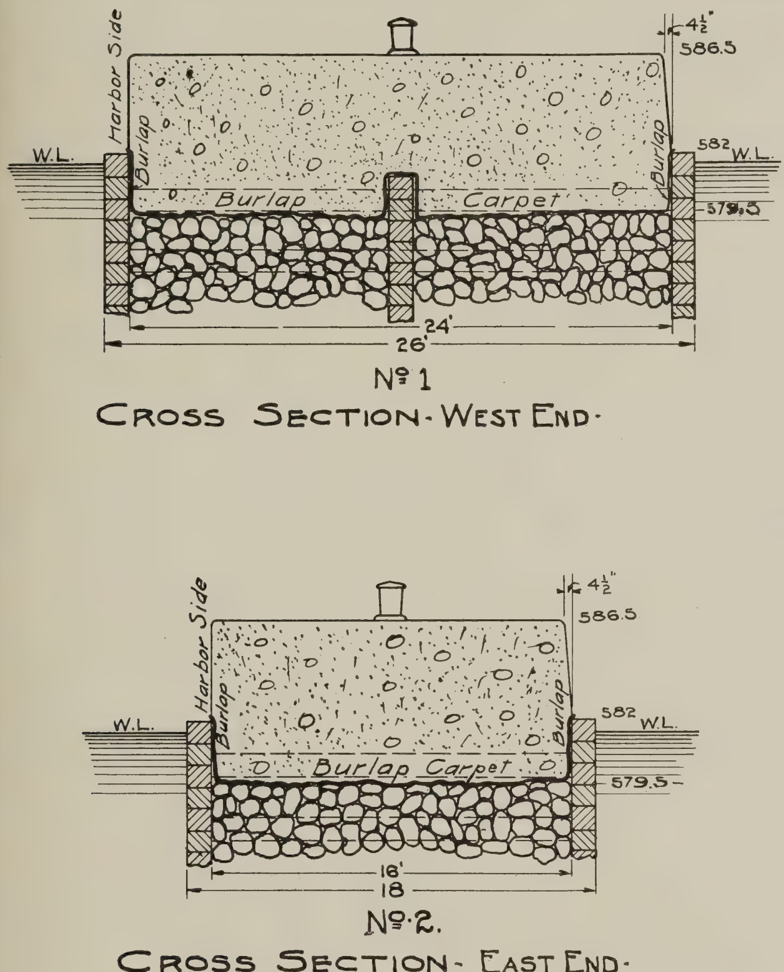

Fig. 4. Section of concrete superstructure, North Pier, Harbor Beach, Mich.

causes, it was decided to remove the stone filling and to place concrete inside the walls to a depth to be always below lowest known low water. The cross ties were left in up to a level with the crib walls. The plastic concrete would assume the shape of all

the irregularities of the foundation and have a uniform bearing over the same, surrounding the cross ties and serving as sway bracing, as it were, for these cross ties. The cribs would thus be strengthened and rendered more rigid.

Definite plans were worked out for the north and south piers, which were yet to be rebuilt, and were approved by Col. C. McD. Townsend, Engineer officer in charge of the district. In December, 1910, a contract was let for rebuilding 1,233 feet of the north pier and 200 feet of the south pier. The cross sections for this work are shown on drawings for north and south piers. The work done may be described briefly as follows:

TEAR-UP.

The superstructure was removed to the required grade which was the same elevation as the top of the footing blocks on the main breakwater, and is above the water surface except at times of very high water. The outside walls of the cribs were leveled to this grade, while the interior wall was not leveled. All cross ties which did not project above the grade of the walls were left in place. The tear-up of the walls was always under control and no timbers below the grade of releveling were loosened in the least. The outside walls were leveled off very easily and quickly. The work consisted mostly in adzing off the timbers to grade. Very few new pieces of timber had to be put on. Old timber from the walls was used in all cases. One man could prepare 100 feet of crib walls in one day. The stone filling was removed to a point 2½ feet below the top of the walls of the cribs. This point was below lowest known low water. At the north end of the south pier, which is much exposed to the full effect of storms and wear from ice floes, the stone was taken out to 4 feet below the top of the crib walls.

MASS CONCRETE.

The stone filling was first covered with a loosely placed carpet of 16-ounce burlap. This burlap was laid loosely over the stone, so that it would assume the shape of the stones and sink into the holes between them when the concrete was placed on it. The crib walls were also covered with burlap from the top down to stone filling, so that no concrete could escape between any timbers. As soon as this burlap became wet it tightened up so that no cement could wash through it. The forms were set up on the walls of the cribs and every precaution was taken to prevent wash of the plastic concrete.

The concrete was put in the forms with a 1-yard bucket and the dumping was carefully done, so as to disturb the water as little as possible. It can be said that this concrete was not washed in any way to injure it. Blocks were made on days when the waves rolled half as high as the forms, yet the water inside the cribs had but a slight rise and fall and the motion was so slow as not to wash the

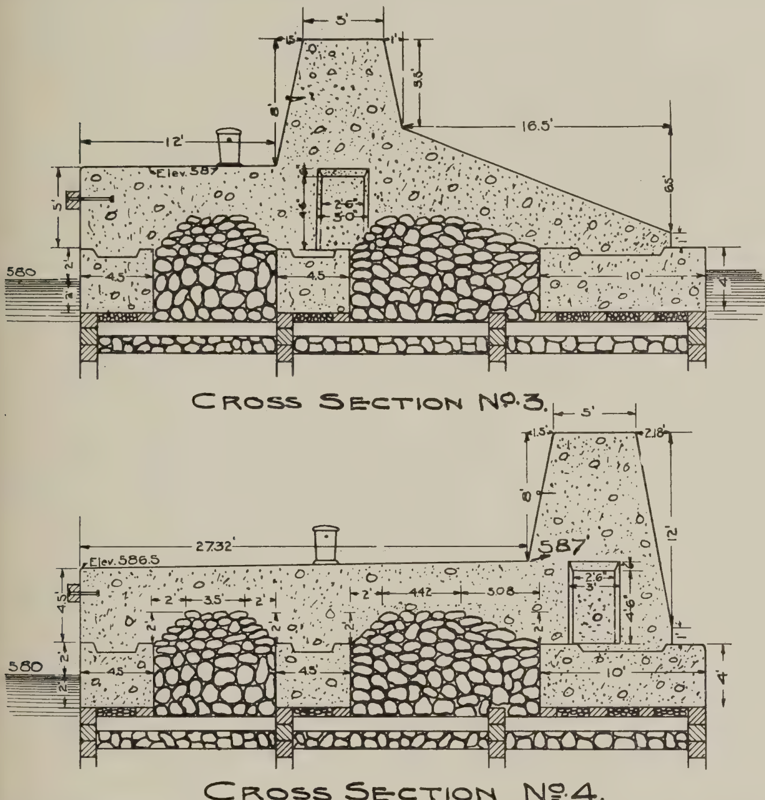

Fig. 5. Cross section of concrete superstructure of the main breakwater, Harbor Beach, Mich., as actually constructed.

concrete. The most exposed part of the concrete was under the forms just inside the crib walls. The waves breaking over the outside wall of the cribs would hit the concrete below the form. Of course, this part of the concrete was protected to a certain extent but not as well as it should have been, and it was exposed to a slap from the breaking wave. Yet this part of the block showed only a rough surface when the forms were removed. The adjacent

block covered this surface. The concrete placed below water surrounded the cross ties left in place. At least two rows of ties in a vertical plane were surrounded with concrete. The result was that the timber cribs were rendered more rigid by the concrete acting as sway bracing on the ties thus surrounded. The plastic concrete also assumed the form of the irregularities of the foundation, and, as the block increased in weight during building, it pressed down in closer contact with the foundation, so that there was less chance of settlement afterwards. The concrete rests not only on the stone filling but also on the timber cross ties of the cribs, thus tying the whole substructure together. The work was done economically and expeditiously, and the results were much more satisfactory than by using footing blocks.

As to cost, the following will give a good idea of the economy of mass concrete. An estimate for doing this work, using footing blocks, and based on the prices received for doing the same class of work as on the main breakwater was as follows:

1,433 linear feet of tear-up superstructure	$9.00	$12,897.00
5,450 cubic yards mass concrete	8.00	43,600.00
2,630 cubic yards concrete in footing blocks	10.00	26,300.00
4,631.3 feet board measure oak fenders	75.00	347.35
17 mooring posts	25.00	425.00
Total		$83,569.35

The cost of the work as built was:

1,432.94 linear feet tear-up (superstructure)	$5.00	$7,164.70
8,322.71 cubic yards mass concrete	6.00	49,936.26
9,642.35 barrels cement	1.25	12,052.94
4,631.3 feet board measure oak fender	75.00	347.35
17 mooring posts	25.00	425.00
Total		$69,926.25

A saving of $13,643.10.

The contract was completed in five months, or, to be more accurate, in one hundred and thirteen working days. If footing blocks had had to be made, at least two seasons would have been required to do the work, and it is doubtful if the work could have been done at the estimate given above, as there was no place convenient to make the footing blocks.

Now that this class of work has been safely carried through to completion and the results have been so satisfactory, it might be well to compare the actual cost of the main breakwater with an

estimate of what the work could have been done for if the latter construction had been adopted. On the drawing marked "Main breakwater as built," are shown the cross sections to which the main breakwater was rebuilt. The section with the parapet in the center was used for all of the breakwater north of the south angle. The section with the parapet on the lake side was used on the portion south of the south angle. The sections show plainly the method of construction.

Cross sections marked A and B show how the concrete might have been placed if footing blocks had been dispensed with. In the two outside pockets, the concrete is placed 4 feet below the top of the walls of the cribs. The crib walls are strengthened by being backed up with concrete. Cross section A is used in making the comparative estimates.

The main breakwater was let in three contracts, and the prices for each item varied some. The prices per running foot were about as follows: $115.00 per foot for the first contract; $108.00 for the second contract; and $120.00 for the third contract. One firm of contractors did the whole work, and the high cost of the third contract was due to lack of competition. Under the head of miscellaneous are included some minor items which were omitted in the first contract and put in later contracts under various items.

Cost of main breakwater:

4,695 linear feet tear-up (superstructure)	$53,926.00
35,975.42 cubic yards mass concrete	312,807.95
14,055.46 cubic yards concrete footing blocks	146,016.32
292,170 feet board measure new timber	12,961.68
157,861 feet board measure old timber	3,218.19
46,156 feet board measure oak fender	2,577.95
92 mooring posts	4,100.00
Miscellaneous	3,879.36
Total	$539,487.45

Estimate of the cost of the work if it had been done along the lines adopted for rebuilding the north pier: The prices for concrete and tear-up are based on the prices bid for the same class of work on the north pier, while the oak fenders and mooring posts are put in at what they cost on the main breakwater. It will be noted that there is no new timber estimated. This item is eliminated entirely, as there is always enough good timber in the tear-up that can be used in leveling up, and the cost of leveling up is so small that it can be included in the item of tear-up. The estimate

is made for cross section shown on drawing A. This section is similar to the one used south of the south angle.

Estimate for mass concrete:

4,695 linear feet tear-up (superstructure)	$10.00	$46,950.00
54,087 cubic yards mass concrete	7.50	405,652.50
46,156 feet board measure oak fender		2,577.95
92 mooring posts		4,100.00
Total		$459,280.45

or, $97.82 per foot.

A saving of $80,207.00.

It will be noted that there are more than 4,000 cubic yards more of concrete in this estimate than was used in the actual building of the breakwater. This additional weight would give stability to the structure.

By this method there would be about 20 per cent less timber to tear up, and 25 per cent less rock to remove. Leveling off would be from 80 to 90 per cent less work, and the making and setting of footing blocks is done away with entirely. On all days that footing blocks could be set, and, by the way, footing blocks could not be set on days when there was much of a sea, mass concrete could be built. It is estimated that 50 per cent more mass concrete could be placed by this method in one season, so that the whole work could have been done in much shorter time. It took five working seasons to do the work, but not much was done the first season, except get organized, and the last season was not a full season's work.

In conclusion, it may be stated that the work done last summer has gone through one winter without the least sign of settlement, though subjected to some severe storms. The work was done in less time than estimated by thirty-two days. The contractors who did the work had never had experience in this class of work before. Their plant was not the best adapted for the work. To those engineers who are doubtful about placing concrete below water, it can be said that there is no danger from wash if proper precautions are taken. It is surprising what little motion there is to the water inside the crib when there is quite a sea rolling outside. Also, that placing concrete below water is no new thing. It has been placed under water successfully for years at various places and under different conditions.

In this class of construction, the work to be done is reduced to

two main items, viz, tear-up and concrete superstructure and all the work of preparing the foundation is above water. The concrete superstructure can be made of almost any cross section to suit local conditions, and the plastic concrete can be so woven into the foundation cribs as to render them more rigid than when filled with stone, and more rigid than with footing blocks resting on the cribs.

Discussion

Maj. CHARLES KELLER
Corps of Engineers; Member American Society Civil Engineers

The writer was connected with the improvement of the harbor of refuge at Harbor Beach from the fall of 1905 to the spring of 1907, witnessed the completion of practically all the work under the first contract and of much of that under the second for the reconstruction of the superstructure of the main pier, and is therefore naturally interested in the subject matter of Assistant Engineer Duffies' paper.

While not directly relevant, it seems worth while to draw attention to the manner in which the intermittent and fragmentary appropriations for the reconstruction of the superstructure of the main pier operated to the disadvantage of the work and of the United States.

The first contract for the reconstruction of part of this superstructure was for work covering a total cost of $200,000 and was let in July, 1903. This sum of $200,000 included all funds available at that time. The annual report of the Chief of Engineers for 1904 does not show an abstract of proposals received under this letter, but it is the writer's recollection that few proposals were received. The unit price of the lowest bidder, $115.51 per linear foot of breakwater, seems relatively high. Progress under this contract was at first slow. Work was not begun until June, 1904, and up to the close of that calendar year the contractors, who were inexperienced at this kind of work, were assembling their plant and learning how to do the work to best advantage. While these contractors subsequently did excellent work, it is probable that had the sum originally available been larger, more bids would have been received, and an experienced contractor, capable of pursuing the work energetically from the beginning, might have been secured and at less cost to the United States. In the calendar year 1904 only 127 feet of superstructure were built, and from the beginning of the contract to June 30, 1905, only fourteen superstructure blocks or 350 feet were completed. Thereafter, however, the work progressed much more rapidly. The contractor's force had now learned its lesson well. During the fiscal

year ending June 30, 1906, forty-four of the large monolithic blocks, a total of 1,100 feet, were completed.

In the meantime $200,000 more had become available, and on August 22, 1905, contract was entered into for work to this amount. The lowest bidders had, on investigation, been found to be of very doubtful standing, yet, because the firm was able to furnish a bond and had not actually failed upon a previous contract, it was decided to be impossible to ignore their bid, which was lower than that upon which the first contract had been based, the unit price now being $108.62 per linear foot.

Apprehension as to the character of the holders of the second contract proved well founded. Up to April, 1906, they did absolutely nothing, not even going so far, in evidence of their good faith, as to procure any considerable part of a reasonably satisfactory working plant, and on April 19, 1906, the contract was formally surrendered, and on the 21st a new contract was made with the holders of the first contract, the basic prices being those of the second contract. Work under the first contract was finished just after the close of the fiscal year 1906, and the transfer of the second contract to the firm that had successfully completed the first one permitted operations to be continued without delay, and the application of the methods and excellent plant already successfully used. The change was decidedly advantageous to the United States, and under the second contract progress was rapid and the work of high quality.

During the fiscal year 1907 progress was excellent, 1,075 linear feet of superstructure were completed, and considerable other work was done. In May, 1907, contract was made for completing the remaining portion of the superstructure of the main pier, the total consideration of the contract being $150,000.00, and the price per linear foot being $119.98. It will be observed that the third contract was let at a higher price than either of the other two. This was due to the absence of competition, caused presumably by the fact that the third contract was relatively small, and that the presence of the contractor for the other two sections on the ground discouraged other contractors from attempting to compete with him. There was every reason to believe that a firm that had been at work for three or four seasons could readily do the remaining small part of the work better and for less money than anyone else, and they seem to have realized and to have taken advantage of this aspect of the case. Appropriations for the work on the main pier were scattered over seven years. A single appropriation of the amount needed would probably have resulted in some saving.

Assistant Engineer Duffies has noticed some of the features of the design of the superstructure of the main pier, and in subsequent work on the north and the south piers changes in details were introduced with gratifying success. Under the first contract for the superstructure of the main pier, adjoining monolithic

blocks were separated by tar paper in the vertical joints between them and the structure as a whole was tied together by steel rails imbedded in the alternating blocks of the series first built and projecting into the mass of the blocks subsequently built to close the gaps in the first series. In the second series this detail was changed by the omission of the steel rails, and the substitution of a recess or sunken panel in the ends of the blocks of the first series. Corresponding projections on the intermediate blocks served to form a bond. The latter detail seems better adapted to accomplish the end in view than the steel rails first used.

Mr. Duffies has mentioned the fact that the surplus stone taken from the old superstructure was deposited on the lake face of the main pier. Though of small size, there was enough of this stone to form a serviceable wave-breaker, and this proved to be of much value, since vessels actually tie to the breakwater when in the harbor, and it is therefore desirable to dissipate the force of the seas as much as possible before they reach the actual superstructure, the form of which was such that some inconvenience is, during even moderate seas, caused by the water that is dashed over the parapet. No doubt, objections may be raised to any design, and that which was actually adopted is no exception. Its form was somewhat complicated and difficult of construction, and the location of the parapet so near the inner edge of the breakwater (width of deck 12 feet only, except for the short length of superstructure near the south end, whose cross section was similar to "Section A") narrows the space available for occupation and, as previously noted, may expose vessels moored to the breakwater to severe wave shocks. Seemingly, it would have been better and cheaper to have used Section "A" throughout. The addition of the wave-breaking rip-rap would have produced a thoroughly satisfactory pier and anchorage, the parapet of which would have been 27 feet or more from the vessels sheltered. Such a change was proposed in the second contract, but disapproved by higher authority and was permitted at the south end only in connection with and because of the light-house there.

As a rule, breakwaters on the Great Lakes protect anchorages and are not used as moorings. In none of them is the superstructure of the cross section used at Harbor Beach main pier. The cross sections of Marquette, Milwaukee, and Buffalo concrete superstructures are of interest in this connection. The Marquette form has no apparent advantage over that adopted at Harbor Beach, though the latter is wider than seems absolutely necessary.

The method of constructing the concrete superstructures of the north and south piers at Harbor Beach is, as has been shown by Mr. Duffies, superior, from every point of view, to the method originally employed there and at other points on the Great Lakes. As Mr. Duffies says, the use of mass concrete deposited in places at and below the water level is not new or peculiar to the Harbor Beach work. The MEMOIRS has cited a case of such use at

Oswego; it was also tried at Buffalo, but with no great success, in 1890 or thereabouts, and has since been used with great success at Superior Entry. The Detroit River tunnel presents a similar case, although perhaps not precisely comparable, and the core wall of the Moline Pool dam is another. The prices at Harbor Beach, even under the latest contracts, seem, however, to be excessive. Six dollars and fifty cents per cubic yard for mass concrete, deposited under the conditions prevailing there, would seemingly have been a remunerative price, and such a price could, judging by experience with the Moline Pool dam, have readily been attained with a hired labor plant.

Mr. B. A. Todt*

Superintendent

When I succeeded Mr. Duffies as superintendent in charge of the Harbor Beach work, about August 1, 1911, the work of constructing a concrete superstructure for the North Pier was well under way. The first monolithic block was molded on June 13, 1911, and by August 1 twenty-one blocks were completed, aggregating 3,022.5 cubic yards of concrete. The entire work, aggregating 8,322.71 cubic yards and consisting of sixty blocks, was completed on September 29, 1911. The work has passed through a hard winter without showing defects, such as cracks or spalls; in fact, is in as perfect condition as on the day of its completion.

Mr. Duffies has amply described the phases of the two modes of construction, one in which footing blocks were used and the other in which they were omitted, mass concrete being substituted.

The advantages of the second plan may be summarized:

1. Time and expense saved in preparing an unreliable and inadequate foundation for the footing blocks, at the most critical stage of the work.
2. Avoidance of injury to the timber substructure, when tearing off the superstructure.
3. Utilizing all available surface for an efficient bearing of the concrete work.
4. Rapidity of construction.

In the plan advocated, the superstructure is removed to a plane above high water, and any damage to the remaining timber work is readily discovered and repaired. The crib ballast is removed with an orange-peel bucket, and the repairing and caulking of the crib walls and laying the burlap carpet to 3½ or 4 feet below water level is accomplished without the aid of divers.

After the ballast has been removed, the defects in the walls are

*Mr. Todt succeeded Mr. Duffies in charge of the Harbor Beach work when the latter was assigned to duty as assistant engineer in the office of the Chief of Engineers, Washington, D. C.

repaired, all holes blocked up, the joints of the crib timbers caulked with oakum, and the burlap laid loosely on the bottom and carried to the top of the walls and secured. The burlap was extended beyond the slopes of the isolated blocks under the forms at the ends of the block, and the lower edge was blocked up to prevent wash from this source. The burlap (16-ounce per yard) effectually pre-

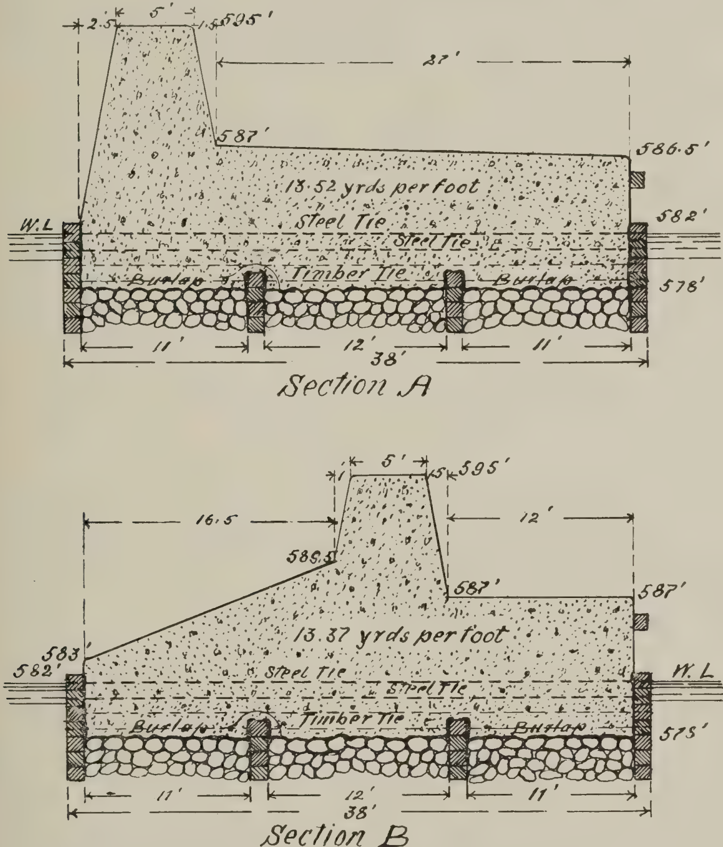

Fig. 6. Modified sections of Fig. 3; main breakwater, Harbor Beach. (See p. 580.)

vented wash, and discoloration of the water on the outside of the pier was rarely perceptible. The caulking of the crib walls, laying the burlap, and assembling the form, took from one to two hours; and the filling of the form of a block of 148 cubic yards place measurement required four to five hours, with one mixer of 1 cubic yard capacity.

I have made special efforts to ascertain the actual cost of the two

principal items of the contract—that is, the removal of the superstructure and the placing of the mass concrete, and find:

Cost of removing the superstructure, including operating expenses of the floating plant and percentage of general expenses per linear foot	$2.9175
Cost of placing mass concrete, including cost of forms, burlap, tarred paper, tie rods for intermediate blocks, operating expenses of floating plant, percentage of general expenses, etc., per cubic yard	3.812

Itemizing the latter, I find:

Cement and gravel per cubic yard in place	$2.6160
Sundries (forms, burlap, etc.)	0.5216
Labor	0.6844
Total	$3.8220

The contractor's investment in the construction plant, amounting to about $19,000.00, is not considered in the above computations; but a liberal allowance was made for depreciation. This plant was purchased from the former contractors, and had been in service during the construction of the Main Breakwater, and while not ideal, answered the purpose.

REMARKS.

Mr. Duffies' views of the construction of concrete superstructures on timber substructures I fully endorse. More substantial and lasting work is obtained at less cost, and the best recommendation I can give is an examination of the work, and a comparison of it with the several plans in vogue.

Referring to Mr. Duffies' proposed sections of the Main Breakwater, I would at increased cost go a step further and attempt to avert the several faults of the present practice of monolithic pier construction.

Fig. 3, sections A and B, show the plan recommended by Mr. Duffies. The footing blocks are omitted, but the core of the crib ballast, between the intermediate longitudinal walls, still remains, an invitation to cracks on the interior of the parapet, when subsequent irregular settlement takes place. I would modify these sections by removing all ballast to 4 feet below high water, remove the two upper tiers of timber ties and substitute steel rods, also cut down the intermediate walls to 3 feet below the above level, and start the monolithic block on the floor so obtained. Timber walls and ties in mass concrete, in my opinion, cut up the block into a number of sections, and when settlement occurs are likely to become the starting points of break-ups. The illustration, Fig. 6, sections A and B, illustrates my ideas. This plan would add about 2 cubic yards of concrete per linear foot of pier, but future repairs would not likely exceed first cost.

The liberal estimate of Mr. Duffies, for the reconstruction of the Main Breakwater according to the sections shown (Fig. 3, sections A and B), can in the light of the actual cost given above be reduced, and I submit the following:

4,695 linear feet of tear-up, at $7.50	$35,212.50
63,477 cubic yards of concrete in place	428,469.75
46,156 feet board measure oak fenders	2,577.95
92 mooring posts	4,100.00
Total cost	$470,360.20

Or $100.18 per foot, which is a saving of $69,127.25 over the actual cost, but an increase of $11,079.75 over Mr. Duffies' estimate. The concrete is increased by 9,390 yards.

Allowing 50 per cent more for the greater width of the main pier, as compared with the width of the north pier, the contractor's profit for the removal of the superstructure would be about 74 per cent, and for the mass concrete fully that amount. The investment in the construction plant is not considered, but on a contract of $150,000.00, a suitable plant could be paid for and a fair profit remain to the credit of the contractor.

Mr. L. C. Schnell*
Junior Engineer

As there is no work of that kind under contract at the present time in the Cleveland District, I will discuss briefly the method adopted in building the concrete superstructure on the West Breakwater in Cleveland Harbor, Ohio, from August, 1897, to the close of the contract in 1904.

DESCRIPTION OF BREAKWATER.

The West Breakwater consists of 106 timber cribs 50 feet in length and ranging in width from 26 to 32 feet with cross walls at intervals of 10 feet C. to C., and one longitudinal center wall. The cribs rest upon a shallow foundation of small riprap, 50 feet wide and about 18 inches deep. The plan of construction was as follows:

The old timber superstructure was removed to a depth of from 2 feet minimum to 3½ feet maximum below mean lake level, and the base for concrete blocks prepared. The lake face was covered with 4-inch white oak plank, extending from a point ranging from 2 to 3½ feet below mean lake level down to the riprap protection at the base of the cribs. This sheathing was securely fastened to the crib timbers with ¾ by 16 inch lag screws, and the concrete blocks were set flush with the outer edge of sheathing, giving a bearing of 16 inches to the outer side of the concrete

*Cleveland District.

blocks. At the beginning of operations we experienced some difficulty in removing the timbers to the proper plane without loosening the timber underneath and below grade. At the suggestion of the writer the contractor had a bolt-jack constructed for starting the bolts in the upper courses of timbers to be removed. This jack was operated by two men in wading boots, and the work was easily accomplished. After all of the bolts had thus been started, a similar device attached to the chain hooks on derrick boom was used in pulling the bolts out with the derrick power. This process was used throughout to the completion of the work with great success. The removal of the old structure was carried on for an average of 150 feet in advance of the blocks. The outer wall was

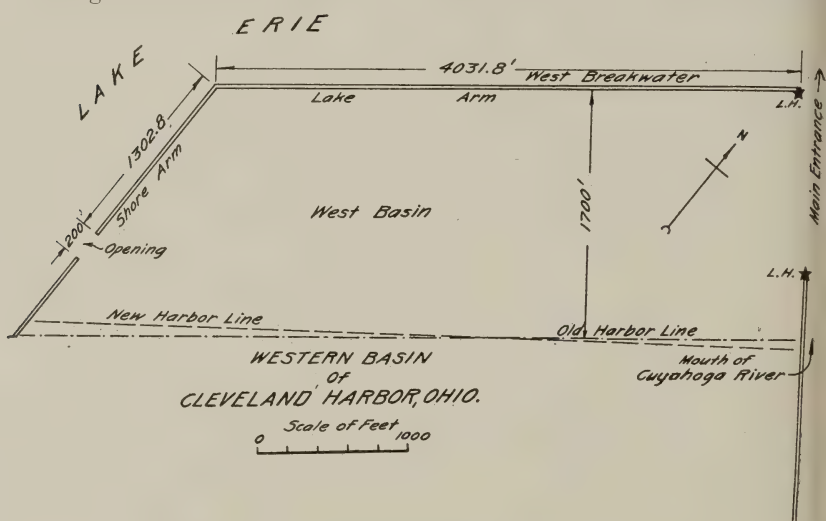

Fig. 7. Western basin, Cleveland Harbor.

left intact, and braced to form a protection for the blocks and concrete set on the harbor and center walls. After this part of the structure had been advanced to 100 or 125 feet the outer wall was removed and the concrete blocks set in place. Then followed the concrete base of the superstructure in its regular order. The blocks for the lake and harbor sides were made uniformly 4 by 4 by 10 feet in length, except at the end of cribs where the length was made to meet the conditions of the adjacent cribs. The blocks for the longitudinal center walls were made 3½ by 4 by 10 feet. Where abnormal conditions in the irregularity of cribs to a horizontal plane were met, divers were employed in following the seam nearest the prescribed depth beginning at the high part of the crib and driving shims at each cross wall and at each corner of cribs upon which elevations were taken and the blocks made to correspond with the grade of the crib. This system obviated the necessity of leveling up the cribs and greatly added to the stability of the

work. Had the contractor adopted this method from the start on all irregular cribs, no matter how slight the difference in level, he would have saved himself thousands of dollars. Pursuing the other course, nearly every block had to be jointed in order to have the required joiner space. Subsequently, under another contract, this method was adopted by the same contractor. Shims were driven at the prescribed depth at each cross wall and at the corners of cribs along the entire work, and elevations taken at each point and the blocks made accordingly to the respective elevations, and when set in place every block fitted nicely and with perfect joints

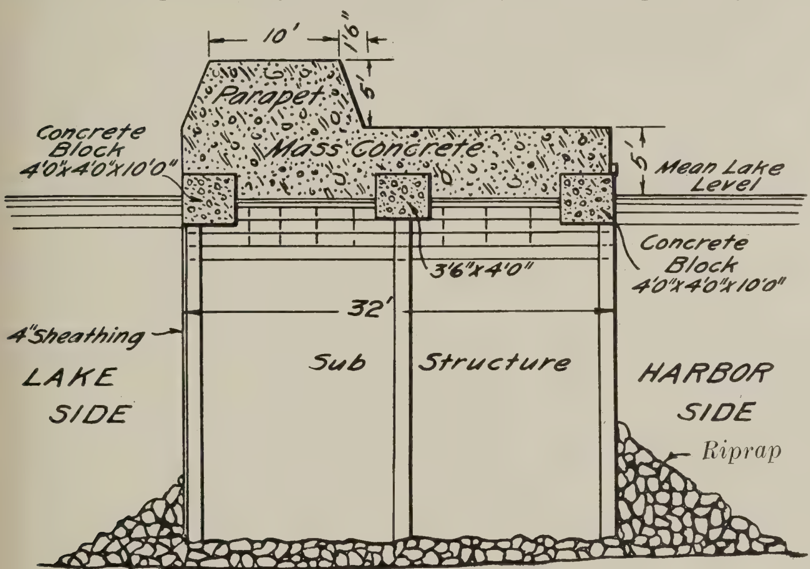

Fig. 8. Cross section of breakwater at Cleveland Harbor, as constructed.

and beds. Care is required in properly numbering each block for its respective position in the work. Had this method been adopted at Harbor Beach the leveling up of cribs would have been unnecessary, besides adding materially to the stability of the structure. I do not concur in Mr. Duffies' idea of building a concrete structure weighing 23 tons per linear foot, supported by the grillage timbers and stone filling of a crib only. It is my candid opinion, based upon past experience, that such a structure should rest upon the outer walls of the substructure to insure the greatest stability. With the superstructure resting upon the interior of filling and open crib work, sooner or later the action of the seas will weaken the tie heads and cause them to give way and the entire structure will collapse. I feel quite interested in this new idea and its ultimate results. To my knowledge, the idea of building concrete structures under water where there is the least particle of agitation has never proved successful.

Mr. G. A. M. Liljencrantz*
Assistant Engineer

The first portion of the paper contains a general description of the method of substituting a concrete superstructure for the old timber superstructure over the Harbor Beach breakwater at an earlier period, the object being obviously to demonstrate the disadvantages in the methods employed at that time, notably the use of facing or footing blocks, as compared with the later method, a description of which constitutes the principal subject of the paper.

Similar conclusions were arrived at by the undersigned, and based on similar observations, in 1908, and a patent was obtained in 1909 which, among other points, provided for the construction of concrete superstructures in form of monoliths in successive sections, built alternately and connected through iron rods, which first served to hold the forms together and later, being left in the work, served as reinforcements. By a special device none of the cross-rods, used as intimated, were allowed to reach the face of the concrete.

Boxes, made of the cheapest grade of lumber and either filled with slag or other cheap filling or left empty, but braced to withstand the pressure from without, were provided for in the patent. These, which I called ''cell boxes,'' serve to save the cost of a considerable amount of concrete, in cases where the cross section is very large, and at the same time, especially when empty, prevent an excessive weight over the old cribwork, which may otherwise become too top-heavy on piers located in deep water and exposed to heavy seas. The cell boxes divide the superstructure into longitudinal and cross walls, all reinforced by the iron rods used for holding the forms together, but are left in the work when the forms are removed.

Plans were made according to these principles, with certain modifications, for the breakwater in Michigan City Harbor, Ind., and built in 1911. The breakwater has the worst exposure to severe northerly storms of any harbor in Lake Michigan—a sweep of about 300 miles.

To place footing blocks and build forms for the mass concrete in a place so exposed to heavy seas would be exceedingly risky, besides which the preparation of the foundation for the footing blocks would have been both slow and expensive, if not entirely impracticable.

As provided in the plans, the formation of the foundation was very simple, consisting merely in the removal of the timber superstructure to, or slightly below, the level of the proposed base of the concrete, leaving the bolts in the side timbers in place, where they would furnish a strong bond with the concrete surrounding them. Then 3 by 12 inch oak planks, with the upper edges dressed and

*Chicago District.

straight, were secured to the side timbers on each side of the work, with the dressed edge in the exact level of the proposed base of the concrete, where this covers the side timbers. The stone filling in the crib was brought up to about 1 foot below the edge of the oak plank, was levelled off with spall and gravel and covered with burlap. Thus the base of the concrete inside of the side timbers was 1 foot lower than that above these.

The forms were made and assembled away from the pier (which work could thus be done in any kind of weather) and held in readiness for a favorable day, when they were placed by means of a derrick on the work, resting on the straight edges of the oak planking and thus secured in a perfect level, without any adjustments or caulking being required, and the filling of the forms then was proceeded with.

The oak plank, besides serving to give the forms an accurate level position, also formed a protection later to the work from drift ice. Steel plates of greater width, say 2 or 3 feet, if used instead of the oak plank, would naturally serve the latter purpose better.

The work at Michigan City Harbor withstood the exceptionally severe weather and storms of last winter without any noticeable damage.

In describing the ''tearing up of the old superstructure,'' it is stated:

''The timber above water was mostly hemlock, which, in late years, it was necessary to use in repairs and was badly rotted, etc.'' During the last forty-one years no hemlock timber has been allowed to be used at or above the low-water level in this district, except in a few cases where temporary repairs were required and where permanent reconstruction was expected within a short period. The fact that it was ''badly rotted'' suggests its unsuitability in such positions.

The statement that the timber from 1 to 2 feet above ''the average lake level'' was ''perfectly sound'' is an endorsement of my contention, contradicted by some, that it is perfectly safe to retain timbers as foundation for a concrete superstructure without going down deeper than to ordinary low-water level (about city datum in Chicago), as this timber will always be wet, there being practically always some slight motion in the water level, and this will prevent rotting. The extreme low water occurs generally in winter when timber is not liable to decay. There is, however, one objection to having concrete-bearing side timbers unprotected at or near the low-water level, viz: the danger of injury by drift-ice, and protection against this should be provided as suggested above. This protection can be easily replaced by new material whenever required. In some of the old work in this district it was found that the floating ice had cut into the timbers near the water level to half of their width, leaving the drift-bolts exposed.

It seems to me hardly advisable to rely too much on the stone

filling as a support for the heavy concrete blocks. The stone filling may settle more or less, as has happened in many instances here, and I believe that the side timbers, which form solid walls from the bottom of the work to the top, should be depended upon for the chief support. As the upper two cross ties in each vertical tier were surrounded with concrete, as described under the heading "Mass concrete," these ties would support the concrete, after this has set, entirely independently of the stone filling, and it would not seem necessary to have the concrete mass fill all the hollows in the riprap.

The irregular bottom produced by such mode of filling would be useless if there should be settling of the stone below, and it would be difficult to determine with accuracy the amount of concrete placed in the work over an irregular bottom.

The comparison between the use of footing blocks and the use of mass concrete, exclusively, shows a positive and considerable saving in both time and expense in favor of the latter method.

In the bill showing the "cost of the work as built" appear two items, as follows: "8,322.71 cubic yards mass concrete," and "9,642.35 barrels cement." Does not the first item *include* cement, and if so, where was the second item (cement) used?

Mr. J. A. B. Tompkins
Assistant Engineer

The method adopted at Harbor Beach, Mich., of constructing concrete superstructure on the crib breakwater without the use of concrete footing blocks, appears to be a very satisfactory solution of a vexatious problem. In building concrete superstructures on old timber cribs the most troublesome portion of the work, and the one involving the greatest expense for the amount of work accomplished, has been the preparation of the foundation. The men have to work in water waist-deep, which is certainly not the most effective manner of working, even though they are provided with wading suits. It is a matter of difficulty to place timber so as to insure a level bed for the footing blocks. The cribs are usually more or less out of level and special wedge-shaped timbers have to be prepared after taking measurements under water. After the cribs have been leveled to receive the blocks, the latter can only be placed in good weather and then by use of a large derrick. Footing blocks have to be made very heavy and breakwaters especially are always in exposed localities.

The use of the crib walls as forms for that portion of the concrete extending under water is a novel idea to the writer at least, and one well worthy of consideration. It is believed that this method could have been used in building concrete superstructure on the Milwaukee breakwater and considerable expense have been thereby avoided.

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*Milwaukee District.

The Milwaukee breakwater incloses a portion of Milwaukee Bay, thereby forming what is known as the "Harbor of Refuge." This harbor was designed primarily to afford a protective anchorage ground for vessels seeking shelter during severe storms. The breakwater was not designed to permit of vessels mooring to it, its purpose being only to check the oscillation of the waves. The original breakwater, the construction of which was begun in 1882, was 7,250 feet long and was composed of timber cribs generally 100 feet in length. For a distance of 600 feet from the shore these cribs were 20 feet in width; thence for 3,650 feet, 24 feet wide, the outer 3,000 feet of the breakwater being composed of cribs 30 feet in width. The cribs were built of 12 by 12 inch timbers with cross ties provided with dove-tailed tenons and were placed on a foundation of stone built on the lake bottom. The original superstructure, which was of timber and of similar construction to the

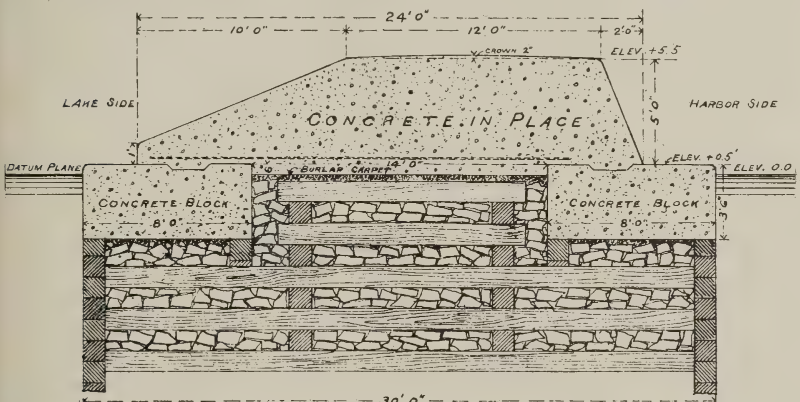


Fig. 9. Milwaukee breakwater.

crib substructures, was carried to a height of about 6 feet above datum, with vertical faces on both sides.

The timber superstructure had become very much decayed, especially over the older portion of the breakwater, and it became imperative to rebuild it in some form. After due consideration it was decided to rebuild the superstructure in concrete rather than in timber, thereby securing permanent work. The general design adopted is shown by Fig. 9, which is a cross section of the superstructure of the 30-foot cribs. This section is fairly typical of the superstructure of the entire breakwater, and is believed to be generally self-explanatory. The footing blocks were 8 feet long, 6 feet 3 inches wide, and 3 feet 6 inches thick, so placed that their upper surface was 6 inches above datum. No decking was used for supporting the footing blocks as was the case at Harbor Beach. On the 20 and 24 foot cribs, the footing blocks rested on the crib walls, and on the interior longitudinal timbers. On the 30-foot cribs special timbers were placed on the cross-ties for supporting

the inner ends of the blocks. While the stone filling of the cribs was carefully leveled under the blocks, no dependence was placed on this filling to carry any portion of the load.

As the breakwater was designed to check oscillations only and not to provide moorage for vessels, it was decided to carry the concrete superstructure to a height of 5½ feet above datum only, except over the 20-foot cribs where the height was 5 feet above datum, and to give the lake face a long, gentle slope. With the original timber superstructure seas striking the vertical walls were thrown upward to a great height, falling on the deck of the breakwater about two-thirds of the way across with terrific force. The gentle slope of the adopted design permits seas to roll over the breakwater with a minimum disturbance, and it is seldom, if ever, that a wave is thrown high into the air. The disturbance produced in the harbor is that due to the falling mass of water only, which produces a surface oscillation that is felt for only a very short distance. So far as stilling effect of the inclosed anchorage basin in concerned, it is believed the form of superstructure used at Milwaukee is fully as effective as is the higher superstructure used at Harbor Beach. It will not, however, permit of vessels mooring alongside of the breakwater. It is needless to say the amount of concrete required is very much less than is the case with the Harbor Beach breakwater.

The method of building this concrete superstructure was practically the same as that used at Harbor Beach, as described by Mr. Duffies. The cribs were leveled to a plane 3 feet below datum, this plane being 581.63 feet above mean tide at New York City. Much of the timber which was removed was perfectly sound; in fact, timber is usually sound that is not more than 1 foot above datum. In removing timber from the old harbor pier at the entrance to Milwaukee River, it was found that white pine timber which had been placed in the pier fifty years before was perfectly sound at 1 foot above datum.

The concrete superstructure on the Milwaukee breakwater was built by contract. Three separate contracts were awarded, but all to the same contractor, Mr. William H. Gillen, of Milwaukee, Wis. The total cost of the entire superstructure was $411,703.47, the average cost per linear foot being approximately as follows:

On the 20-foot cribs, $41.00; on the 24-foot cribs, $53.50; on the 30-foot cribs, $64.00.

The price for timber used in leveling cribs under the first contract was $40 per thousand; under the second contract it was $50 per thousand, and under the third contract $60 per thousand. The contractor had evidently learned that the placing of timber under water was very expensive and that his original price did not fairly cover it. The total amount of timber used for leveling was about 182,000 feet board measure, the average cost being about $47.50 per thousand in place. The total cost of leveling the cribs, exclusive of the cost of the new timber, was about $59,000, an average

of about $8.10 per linear foot for the entire breakwater. The average contract prices for concrete blocks in place was about $9.37 per cubic yard. The average price for the mass concrete was about $8.43 per cubic yard. The work was begun under the first contract on June 10, 1903, and finally completed under the third contract on October 10, 1908. The total actual time required for rebuilding the superstructure was 668 days.

While it is probable that the method adopted at Harbor Beach would have resulted in a considerable saving in cost, the general appearance of the work would not have been as good as it is now, except by modifying the Harbor Beach method. The cribs composing the Milwaukee breakwater had settled badly out of line, while under the method adopted the alignment of the concrete superstructure proper was kept well nigh perfect, all irregularities in the alignment of the cribs being taken care of in the footing blocks, which were long enough to permit of shifting the position of the mass concrete so as to maintain alignment. However, it is probable that a footing could have been obtained over the entire breakwater by using the old crib walls as forms carrying the concrete up to about 6 inches above datum. On this mass of concrete the superstructure could undoubtedly have been built and good alignment maintained. This method would have required a larger amount of concrete than was actually used, but it is probable that the total cost of the work would have been less.

On the Great Lakes, breakwaters and their protective structures have usually been built in years past of timber, this being the cheapest material available. While these structures have served their purpose admirably, timber has a comparatively short life above water and consequently renewals of the superstructure, about every fifteen years on the average, have been necessary. As long as timber was cheap and abundant, these renewals did not become very serious. In the case of an ordinary pier or jetty, say from 20 to 24 feet in width, the cost of rebuilding timber superstructure at harbors in the Milwaukee district would rarely exceed $7.50 or $8 per linear foot, but with the increasing cost of timber as well as labor, the cost of rebuilding superstructure in timber has become about $10 per linear foot. Of course, in renewing the superstructure of a breakwater where the cribs are much wider, this cost will be much higher, possibly as great as $20 per linear foot. These costs are based upon doing the work by hired labor and using the Government plant. If the work is done by contract, it will probably cost from 25 per cent to 50 per cent more.

All the timber used of late years in the Milwaukee district has been obtained from the Pacific Coast. This timber is Washington fir, and so far as strength and durability is concerned is very satisfactory. It is relatively cheap, the average cost of the timber delivered at points on the west shore of Lake Michigan, including inspection and unloading, being about $25 per thousand feet board

measure. This includes the freight from the Pacific Coast, which averages about $10 per thousand feet board measure. It must be borne in mind, however, that for the greater part of the distance the Government rate is only 50 per cent of the commercial rate, this being one benefit the Government has obtained from the extensive land grants made to the trans-continental railroads. Even with timber at this price and by doing the work with hired labor, it was still found that the cost of rebuilding a timber superstructure was about $10 per linear foot.

It has therefore become the general practice in the Milwaukee district to rebuild the superstructure on such works as are known to be permanent, in concrete. The cost of the concrete superstructure is probably 40 to 50 per cent greater than it would be in timber, but when the permanency of the concrete work is con-

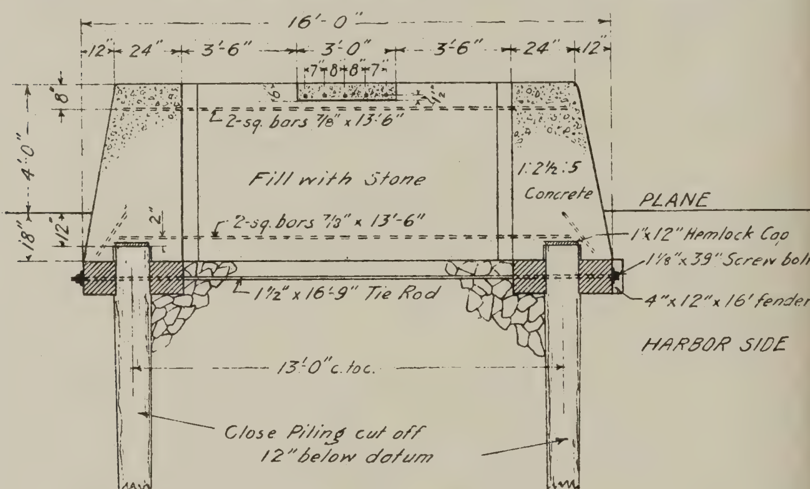


Fig. 10. Menominee pile pier.

sidered, it can readily be shown that it is true economy to build in concrete rather than in timber.

Many of the old harbor structures in this district are pile piers or jetties. The first pile piers to be built consisted of two rows of round piles about 13 feet apart. The piles were driven close together in a row, provided with binders and ties, and a superstructure of 12 by 12 inch timbers built above the water. Fig. 10 is a cross section of such a pier at Menominee, Mich., and shows the method adopted for providing it with concrete superstructure. The old round piles were cut down 1 foot below datum. New wales and binders of 12 by 12 inch timber were provided, and the pile walls held together by iron tie-rods placed about every 8 feet. The round piles were capped with 1 inch hemlock boards and 1 inch hemlock strips were placed along the sides of the piles where they project above the wales, thereby boxing in the pile heads to

prevent leakage of the concrete while being deposited. Concrete walls were then built as is shown in the illustration, so that the pile heads were embedded in the concrete. The superstructure, which is cellular in type, was built in sections 24 feet in length and cross walls 12 inches thick were provided every 8 feet. These cross walls had four reinforcing bars embedded into them, the bars extending into the main side walls of the superstructure. It will thus be seen that the pile walls are held in place not only by the timber wales and iron tie-rods, but by the concrete superstructure as well. Even if the $1\frac{1}{2}$-inch tie-rod passing through the wales should fail, the pier could not spread. The pier is provided

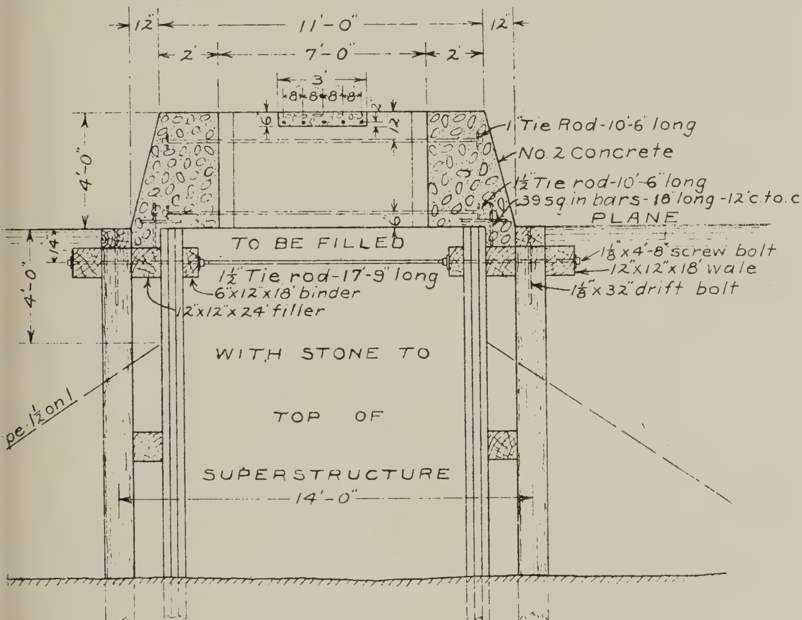


Fig. 11. Racine pile pier.

with a walk consisting of reinforced concrete slabs 3 feet wide and 6 inches thick. These slabs are molded away from the site of the work, at any convenient place, and after becoming thoroughly hard are lowered into position by a derrick, recesses being provided in the cross walls to receive them. The slabs are molded in lengths of 7 feet 10 inches each. The entire pier, including the superstructure, is filled with rubble stone. This form of construction has proved to be very satisfactory and has been built at a cost of not to exceed \$15 per linear foot. This price includes cutting down of the old superstructure and refilling the pier with stone.

Fig. 11 illustrates the method employed in this district in constructing a new pile pier with an original concrete superstructure.

It will be noted that advantage is taken of the cross walls, as in the previous case, to provide an additional tie for the pier. In this case, however, the sheet piling only is held by the concrete superstructure. All the new pile pier work done in this district is provided with sheet piling on each side. Several hundred feet of this kind of pier are being built at Manistique Harbor, Mich., and at Racine Harbor, Wis. The work is being done by contract and the cost of the pier, which, of course, includes the piling, stone filling, etc., averages about $38 per linear foot.

While, as has been previously said, the timber cribs heretofore built have been satisfactory and while timber below water does not rot, it is found that these cribs are subject to deterioration. On the Great Lakes, of course, there is no trouble experienced from the teredo, but sand is driven back and forth through the joints in the timber work gradually cutting the timber away, so that in the course of years the entire structure becomes more or less loose and is subject to racking by the waves. It would seem that some better form of construction should be adopted than cribs. Furthermore, it does not seem consistent that the General Government should set aside forest reserves and endeavor in every way to protect and conserve our forests and at the same time place millions of feet of some of the finest timber in the world under water where stone will answer the purpose fully as well. At many harbors on the Great Lakes, notably those on Lake Erie, the breakwaters are being constructed of rubble stone. This form of construction, however, requires a covering of large stone weighing about 10 tons each. On Lake Michigan, unfortunately, stone of this size can not be obtained and the cost of bringing it from Lake Erie would be prohibitive. In the Milwaukee district, of late years, newer breakwater construction has been of reinforced concrete caissons, built from designs of Lieut. Col. W. V. Judson, Corps of Engineers. A description of these caissons might be interesting, but it is hardly germane to the matter in hand and, as Kipling says, "That is another story."

At Manistique Harbor, Mich., it became necessary to extend the old breakwater. It was originally intended to have built this extension of reinforced concrete caissons. There were local difficulties, however, which were finally concluded to be prohibitive. The caissons could not be built at Manistique, and the cost of towing them to Manistique from the nearest point at which they could be built would be entirely too great, to say nothing of the risk involved. It was therefore decided to build the extension with timber cribs resting on stone foundation, all cribs to be provided with concrete superstructure.

Fig. 12 illustrates the method finally adopted. The cribs were built in the usual manner and then, in accordance with plan proposed by Maj. Chas. S. Bromwell, Corps of Engineers, footing blocks were molded in place on the crib as it lay floating at the construction dock. In this particular case the cribs were so shal-

low that the weight of the footing blocks overcame the buoyancy of the timber and the cribs would sink unless provided with additional buoyancy. This was obtained by constructing water-tight boxes which were lowered into the pockets of the crib in sufficient number to insure its flotation. The crib was then towed out to the site of the work and sunk in place by the admission of water into the flotation boxes. When the crib was settled on to the foundation, the other pockets were filled with stone, after which the flotation boxes were pumped out and removed from the crib, to be used again in the next one. The pockets occupied by the flotation boxes were then filled with stone. This method provided a concrete foundation reaching about 6 inches above datum on which mass concrete forming the superstructure could be formed. All timber was at least 6 inches below datum. About 400 feet of the breakwater extension at Manistique has been built in accordance with this plan, which has proved very satisfactory.

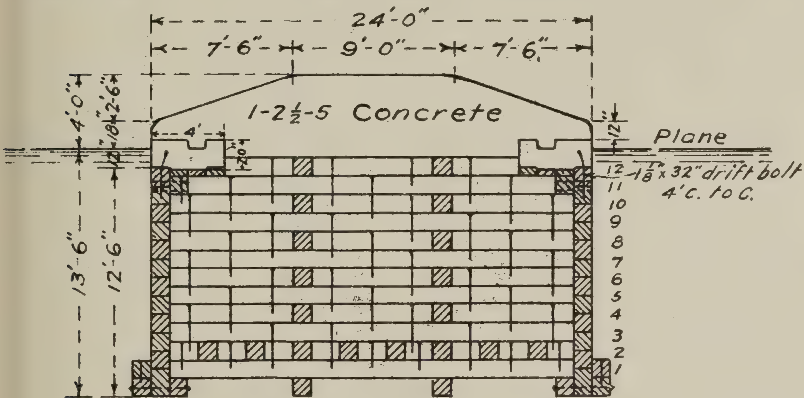


Fig. 12. Manistique breakwater.

No great difficulties have arisen during the progress of the work and strange to say, the original plan, heretofore untried, has worked out without a hitch and without any material modification. The Manistique breakwater is being constructed by contract at an average cost of about $100 per linear foot, this cost including stone foundation, cribs, and superstructure. The timber, which is Washington fir, was purchased by the Government on the Pacific Coast, and delivered to the contractor at Manistique. The saving thereby effected is probably not less than $6 per thousand feet board measure.

Mr. J. H. Darling*

Assistant Engineer; Member American Society Civil Engineers

The question of the height of the line of no decay in timber with

*Duluth District.

reference to the water surface is one of practical importance, and I am glad to see the facts observed by Mr. Duffies bearing on this point in his work of tearing up the old timber at Harbor Beach, and to note that they accord well with facts observed on Lake Superior. On this lake observations taken in recent years by the Duluth office indicate that the line of no decay in pine timber is above the low water elevation, and with rare exception is somewhat above the mean level of the lake.

In cribs at the Duluth Harbor entrance which had been in place about twenty-seven years, the writer found no decay below the elevation +1.0 (referring to low water datum, this datum being about 0.6 foot below the mean lake level), and generally none below +1.5 feet.

At the Superior Entry, an examination in May, 1910, by Mr. M. W. Lewis, Junior Engineer, of the cribs that had been in place about forty years showed no decay lower than +0.4 foot referred to the same low water datum, and but little below +1.0.

At Quebec Harbor, Michipocoten Island, at the east end of the lake, a very old and abandoned wharf was observed by the writer in which the piles of Norway pine were still in place, but much decayed at the top. The lowest decay found on these piles was at about +1.0. Many of the piles were decayed from the top down to about +1.5. They were sound below these points. This was in a sheltered harbor.

At Ashland, Wis., the writer last month observed an old railroad trestle that was abandoned, decayed, and going to pieces. The superstructure of ties, caps, stringers, and braces was generally much decayed, and also the piles, which were of pine, down to about +1.5. Some of the piles showed decay down to about +1.0, but below +1.0 they all appeared perfectly sound. The line of decay was very well defined. This was at the harbor front, on a large bay connected with Lake Superior.

A year ago the city of Superior was rebuilding a highway trestle over the swamp adjoining Nemadji River at Fourth Street, which had stood sixteen years and was decayed and unsafe. City Engineer E. B. Banks stated to the writer that "No decay of the piles is found up to about 1 foot above datum." His plan was to cut off the old piles at datum and use them to support a new trestle. The datum referred to is the Government low water datum already mentioned, and the water in the swamp stands at about the lake level.

The old cribs at the Duluth entrance date back to about 1872, and those at the Superior Entry to the sixties, and the subsequent period has included a number of extreme fluctuations of the lake, including low stages in 1877-1879, 1886-1887, and 1891-1892, as well as high stages in other years, and it is reasonable to presume about the same average stage in the future, with a recurrence of low and high stages more or less similar to those in former years, and that the conditions affecting decay will not be greatly different.

In view of the facts mentioned, it has been considered safe to carry the timber substructure up to about 1 foot above low water. This facilitates the placing of mass concrete, avoids the necessity of footing blocks, which tend to weakness, and lessens cost as shown in Mr. Duffies' paper.

The cribs at the seaward end of the new breakwaters at Superior Entry are at +1.0 foot, and the cribs planned for a breakwater extension at Marquette are at +1.0. Current practice among engineers and architects has usually been to stop timber foundations somewhere below low water, at various depths without any uniformity. In recent years there has been a change in this respect in this locality, and a tendency to go higher with the timber. For examples, at the steel ore dock No. 6 of the Duluth & Iron Range Railroad at Two Harbors, built a few years ago, the bearing piles were cut off at —3.32 (when referred to the Government low water datum). At the Great Northern Railway steel and concrete ore dock at Allouez Bay, Superior, completed in 1911, the top of the crib substructure and the bearing piles is 1 foot below low water datum. At the No. 1 new steel ore dock at Two Harbors, completed this year, the pile cut-off is at —0.82 (referred to same datum) and the concrete reaches 1 foot below the top of the piles. At the L. S. & I. Ry. iron ore dock of reinforced concrete now nearly completed at Presque Isle, near Marquette, the bearing piles are cut off at low water datum.

The mean lowest monthly level of Lake Superior is very nearly at 0.0 or datum, and the mean highest monthly level is +1.1. The low stage occurs in winter and early spring, when the agencies of decay in this climate are quiet or nearly so. When these agencies are most active, which of course is in the summer season, the stage of the lake is highest, and capillarity keeps the timber wet for some distance above the water surface.

Mr. E. J. Duffies
Assistant Engineer; Member American Society Civil Engineers

The writer is very much gratified that his article has brought out as much discussion as it has. His object was to present this last method of construction to the Corps of Engineers as a substitute for the old footing block method, which has been in use for so many years. He believes that the mass method of construction is less expensive, more rapid, and more substantial than the footing block method. He is also aware that there is a prejudice among some engineers against placing plastic concrete below water. This is one of the things that is unexplainable, in view of the fact that concrete has been placed below water in a loose state for years, and if done under proper supervision the results are excellent.

In reply to Mr. Schnell's statement that no structures have been built under water with good results where there was the least particle of agitation, I may say that concrete has been placed under

water where there was some slight movement. If the concrete is placed with a bucket, there is some slight agitation of the water, caused by the lowering, dumping, and raising of the bucket, if a bucket is used. Concrete has been placed below water in this manner for over twenty years. In the proceedings of the Institute of Civil Engineers, Vol. LXXXVII, pages 231-237, Mr. Wm. Kidd, Associate Member Institute Civil Engineers, gives a very interesting article on placing subaqueous concrete in the sea at Skinningrove, on the Yorkshire coast about 8 miles south of the River Tees, England. The bottom of the ocean was very irregular and it was necessary to obtain a level bottom for the pier that was to be built. The usual way was to level off the bottom with large bags of concrete or with concrete blocks. In this case it was decided to level off the bottom with plastic concrete deposited by means of buckets. A diver was employed to direct the dumping of the bucket and to level the concrete. It is only necessary here to say that Mr. Kidd's article reports that the work was fully completed in 1891 and was most satisfactory and economical, and that a close inspection disclosed the subaqueous concrete to be hard and sound throughout. Mr. Kidd states that the concrete cost only 12s. 9d. (about $3.02) per cubic yard.

In 1895 and 1896 Major Hoxie, Corps of Engineers, U. S. A., constructed the foundations of the lock at Herr Island on the Alleghany River, under water. A bucket was used in depositing the concrete into the forms. Natural cement was used in this case.

It is stated that over $22,000 was saved over the original estimates for this work. An account of this work may be found in the Annual Report, Chief of Engineers, U. S. A., Vol. III, 1895, page 2410, and Vol. IV, 1896, page 2200.

In the fall of 1901, the writer superintended the placing of about 400 cubic yards of concrete in 22 feet of water at the west end of the north pier of the Duluth Canal. The forms were made from the old decking taken from the old wooden north pier. The forms were lined with cotton ducking to cover the cracks between the planks. The concrete was placed with a bottom-dump bucket. The current was very strong at this point through the canal. The success of this attempt had much to do in deciding to build the substructure of the piers at Superior Entry in concrete. The success of the work at Superior Entry is well known.

The concrete surrounding the tubes of the Detroit River tunnel was placed by means of trémies, but it was placed in over 40 feet of water in the open river, which has quite a strong current.

It is to be regretted that Mr. Schnell did not give figures to show the saving obtained in making the footing blocks to fit the foundation over the cost of leveling the foundation. At Harbor Beach the cribs were so out of plumb and the timber came up in so irregular a manner that the work of molding footing blocks to fit the foundation would have been great, and the blocks in some instances would have been of too great a depth to handle with a derrick.

It is to be regretted that Mr. Liljencrantz gave no figures as to the cost of the work at Michigan City. The writer has found that the placing of cells in mass concrete for this class of work is of doubtful economy.

The amount of concrete saved does not pay for the labor and time of filling the cells with stone. A second operation is introduced along with the concrete work, and a contractor generally prefers to do the one operation, especially where space and plant is limited as it is in building a breakwater in the open lake.

The placing of oak plank at grade to take the place of leveling off of the outside walls is a good idea if one prefers it. It is doubtful if it is more economical. If there is much danger from floating ice wearing away the walls of the cribs, it would seem to be better to place steel plates at once on those walls.

In regard to the question raised by Mr. Liljencrantz as to whether the 8,322.71 cubic yards of concrete included the 9,642.35 barrels of cement, it is stated that it does not. Colonel Townsend preferred to buy the cement from the contractor. The idea was a most excellent one, as it allowed the Government to use as much or as little cement as conditions required. The contractor was paid for what was used. The ''concrete'' includes the aggregate, labor, etc. The actual price per cubic yard paid for the concrete was therefore $7.45 per cubic yard.

The writer agrees with Mr. Todt in most cases in the use of tie rods and the removal of the wooden ties. Where the wooden ties are securely fastened to the walls and the dovetails are in good state of repair, it is not believed necessary to remove them. They help reinforce the concrete, beside helping to distribute the weight of the concrete to the walls and act as transverse bracing to resist vibration. On the other hand, the tie rods serve to tie the walls together and reinforce the concrete and do not break the interior of the cribs into pockets.

Mr. Todt's analysis of the cost of the two principal items of the work is most edifying. The cost for removing the superstructure, according to his figures, is $2.92 per foot. The Government paid $5.00 per foot. The cost for concrete per cubic yard is $3.82. The Government paid $7.45. A profit ranging from 70 per cent to 90 per cent on the money invested is very encouraging to any one doing the work.

It is to be hoped that Mr. Tompkins can be induced to write an article giving the cost of the piers built of reinforced concrete caissons, and of the novel construction being used at Manistique.

The writer has always been of the opinion that the cost of concrete caisson method was unduly high, and from the nature of the construction it was impossible to secure a uniform bearing for the caissons on the foundation, and for this reason settlement will cause cracks in the caissons in the same manner as settlement of timber cribs will cause distortion in them.

In regard to the alignment of the superstructure of the break-

water at Harbor Beach, the writer started out to get a good alignment, as was done at Milwaukee, but soon gave it up. The breakwater had settled so out of line that several distinct tangents would have had to have been adopted in order to keep the superstructure on the cribs. It was therefore decided to set the footing blocks as nearly to the general outline of the breakwater as possible, and the mass concrete, of course, followed the footing blocks. The result was not bad. If one is in line with the breakwater it looks like an immense snake in motion, but a side view is all right. On the piers, on which mass concrete alone was used, no attempt was made to rectify the alignment, but the superstructure followed the irregularities of the substructure.

Some criticisms have been made that the concrete was not placed on the outside walls of the cribs. The first plans were drawn with the concrete on the walls, and the writer was much in favor of that method. But, for other reasons, it was decided to place the concrete as shown in the sketches attached to this article. For constructional reasons, the method as adopted was much more convenient.

Major Keller intimates that the price paid for concrete on the last contract was higher than it should have been. In answer to this, the writer would refer to the actual cost of the two principal items of the work in Mr. Todt's discussion. It will be noted that the profit runs from 70 to 90 per cent of the actual cost of the work. The writer believes that this class of work can be done at a very fair profit for much less than it cost the Government, with conditions existing as they are at Harbor Beach. Then, again, it will be noted that a block of about 148 cubic yards was built in one day. A block containing 200 or 225 yards could have been built as easily, thus increasing the rapidity of the construction.

Factors Affecting the Safe and Economical Operation of Boats in a Restricted Channel in the Hudson River

From investigations and observations made by Lieut. R. D. BLACK, Corps of Engineers, assisted by Mr. W. P. BENJAMIN

An examination of the existing published data on this subject yielded the following information:

a. From experiments in a model tank, using models of steamers of the Hudson River Day Line, length 396 feet, beam 47 feet, draft 7 feet 6 inches, Prof. Herbert Sadler, Dean of the Department of Naval Architecture of Michigan University, has deduced a set of curves showing for that type of boat the relation between indicated horsepower and speed for a number of depths (Plate II).

b. From a set of observations made by the United States under the supervision of H. N. Babcock, United States Assistant Engineer, on screw steamers operating in a relatively deep channel near Sandy Hook, N. J., the data contained in the following table were obtained.

Draft of vessel at pier.		Draft in channel.	Depth of water in channel.	Squat.	Velocity of vessel in miles per hour.
Bow.	Stern.				
24.6	25.8	29.2	31.7	3.4	18.9
24.0	26.0	29.0	32.3	3.0	15.2
29.1	29.5	32.1	35.8	2.6	17.9
24.3	25.2	26.7	33.3	1.5	15.2
22.2	24.7	25.8	37.7	1.1	14.5

c. Other important data bearing on these subjects have been obtained from researches of Naval Constructor Isherwood, R. E. Froude, Capt. A. Ramoussau, and Sir William H. White, and from the report of the Board of Engineers on Deep Waterways. A search for further data on this subject, carried on under the auspices of the American Society of Civil Engineers, failed to produce much other information applicable to conditions on the upper Hudson.

In order to obtain at first hand data on which to amplify and verify relations shown theoretically to exist between draft, speed, power, and channel cross section, a series of observations were carried on in the Hudson River during the months of September and October, 1911.

Two points for observation were selected, viz, at Bogart Island, 2 miles below Albany, N. Y., and at West Point, N. Y. At the first point the channel conditions were as follows: Channel depth, 12 feet; average width of river, 770 feet; cross-sectional area at lowest low water, 5,700 square feet; for each foot of increased depth produced by tidal action, 770 square feet were added to cross-sectional area. Steamboats were timed over a course 400 feet long. At the second point the channel depth was 135 feet and the cross-sectional area was so great as to have no influence on speed or squat.

Observations were taken on the steamers of the Hudson River Day Line *Hendrick Hudson, Robert Fulton,* and *Albany.* Readings were taken of the changes in water elevation, of the speed of the vessel, of the current velocity, and of the elevations of a predetermined point on the moving boats. From these readings and from measurements made at the dock just before the vessel's departure, the squat was computed. The size and character of the swells were recorded as far as practicable. The officials of the Day Line cooperated by running at various speeds asked for and endeavoring to furnish a record of the power used. Available time and funds did not permit of as much work as could have been desired, and it was seldom possible to get more than one set of observations under exactly similar conditions. Consequently, the more general relations deduced are not thoroughly substantiated.

Without giving here in full the results of the field work, of which the observed data are recorded in Plate I, the following somewhat striking observations are selected as being especially typical and interesting:

a. In about 135 feet of water, at a speed of 22.5 miles per hour, the steamer *Hendrick Hudson* squatted 0.99 foot at her stern, and her swell was practically inappreciable at a distance of 1,000 feet.

b. In the same depth, this steamer, at a speed of 11.5 miles per hour, squatted 0.15 foot at her stern.

c. In 17 feet of water, at a speed of 8.88 miles per hour, the squat of the *Robert Fulton* was 0.55 foot, and the maximum height of the swell was 2.5 feet at about 250 feet from the vessel.

1	2	3	4	5	6	7	8		9	10		11	12	13	14		15	16	17
Date. 1911	Place	Name of Steamer	Speed of Steamer Relative to shore Miles per hour	Current Velocity Ms. per hr.	Speed of Steamer Relative to water Miles per hour	Depth of water along axis of Steamer	Drop from Normal Elevation		Tide Gage Rdg Before effected by Steamer	Tide Gage Rdgs as Steamer Passes		Total difference in elevation (in Feet)	Total fall from normal elevation (in Feet)	Wave height (Max.)	Instant of passage		Instant of Highest Water	Instant of Lowest Water	Approx. dist. of axis of steamer from Sta. A.
							Bow	Stern		Mx.	Min.				Bow	Stern			
9-25	B.I.	1	7.79	-0.76	7.03	15.50	.65	.75	4.50	6.0	2.4	3.60	2.1	Same as Column 10 (Max.)	8-46-10	8-46-30	8-46-40	8-46-30	250.00′
26	"	2	11.10	-0.69	10.41	15.70	1.46	2.20	4.60	7.0	4.0	3.00	0.6		-44-10	-44-30	-44-40	-44-30	"
27	"	1	9.74	+0.35	10.09	15.00	1.36	1.60	4.60	5.0	4.0	1.00*	0.6		-45-0	-45-20	-45-30	-45-20	"
27	"	3	10.20	0.00	10.20	15.75	★	1.35	4.30	4.4	4.3	.10*	0.0		-43-10	-43-30	-43-40	-43-30	"
28	"	3	9.74	0.00	9.14	14.40	.90	1.00	4.50	4.5	4.2	.30	0.3		-45-20	-45-50	-46-00	-45-50	"
28	"	1	10.45	+0.32	10.77	12.60	1.65	3.10	2.50	3.7	.3	3.40	2.2		-42-50	-43-10	-43-20	-43-10	"
29	"	1	10.45	+0.32	10.77	14.60	1.13	2.40	4.10	5.0	2.22	3.80	1.9		-47-30	-47-50	-48-00	-47-50	"
29	"	3	10.90	-0.45	10.45	13.70	1.25	2.62	3.70	5.5	1.5	4.00	2.2		-45-10	-45-30	-46-40	-46-30	"
30	"	3	9.10	+0.38	9.48	13.90	1.31	3.25	2.80	4.5	.8	3.70	2.0		-50-40	-51-0	-51-10	-51-00	"
10-2	"	1	10.20	+0.85	11.05	12.40	.25	3.15	2.15	3.7	.5	3.20	1.6		-47-40	-48-0	-48-10	-48-00	"
3	"	3	9.10	+1.10	10.20	12.60	.85	1.25	2.85	3.7	2.5	1.20	0.3		-46-10	-46-30	-46-40	-46-30	"
5	"	3	8.52	-1.10	7.42	13.20	.75	.95	3.90	4.0	3.5	.50*	0.4		-47-20	-47-40	-47-50	-47-40	"
6	"	1	10.20	-1.16	9.04	13.50	.45	1.25	3.50	3.5	1.1	2.40	2.4		-46-30	-46-50	-47-00	-46-50	"
10	W.P.	1	20.50	-1.08	19.42	135.00	.34	.48	2.80	2.8	2.8	0.00	0.0						1000.00′
10	"	3	23.00	-0.51	22.49	"	.19	.99	2.00	2.0	2.0	0.00	0.0						"
11	"	2	21.00	+0.80	21.80	"	-.44	.16	2.90	2.9	2.9	0.00	0.0						"
11	"	1	16.50	-0.49	16.01	"	.17	.45	2.75	2.75	2.75	0.00	0.0						"
12	"	3	12.00	-0.46	11.54	"	-.10	.15	2.80	2.8	2.8	0.00	0.0						"
12	"	2	12.00	-0.16	11.84	"	.15	0.00	2.90	2.9	2.9	0.00	0.0						"
13	"	3	18.00	0.00	18.00	"	-.12	.48	1.65	1.65	1.65	0.00	0.0						"
13	"	1	13.00	0.00	13.00	"	.08	.15	2.90	2.9	2.9	0.00	0.0						"
20	L.A.W.	1	10.90	-2.02	8.88	17.00	.05	.56	6.90	8.0	5.5	2.50	1.4		8-49-0	8-49-20	8-49-30	8-49-20	250.00′
21	"	2	11.90	-2.02	9.88	17.00	.22	1.02	6.90	8.1	4.5	3.60	2.4		-50-50	-51-20	-51-20	-51-10	"
23	"	1	12.50	-1.70	10.80	17.50	1.25	2.05	7.50	10.0	7.0	3.00	0.5		-45-10	-45-40	-45-40	-45-30	"

Column 2- B.I.= Bogart Island
" 2-W.P.= West Point
" 3- 1 = Fulton

Column 3-2= Albany
" 3-3= Hudson
Squat is given in Column 8

Column 5-(-) sign indicates a current adverse to progress of steamer.
" 5-(+) " " " " adding " " " "
" 8-(-) " " " rise above normal elevation.

★Column 8 = Buoy marking position of bow was lost. *Column 11 = Result doubtful. Time given in Col's. 14-15 & 16 is correct to nearest 5 seconds.

Plate I. Data obtained from the investigation of the squat of vessels, Hudson River.

d. In 15.5 feet of water, at a speed of 7.03 miles per hour, the squat of the *Robert Fulton* was 0.75 foot and the height of the swell was 3.6 feet.

e. In 12.4 feet of water, at a speed of 11.05 miles per hour, the squat of the *Robert Fulton* was 3.15 feet.

f. In 14.4 feet of water, at a speed of 9.14 miles per hour, the *Hendrick Hudson* squatted 1 foot, using 350 horsepower.

g. In 13.9 feet of water, at a speed of 9.48 miles per hour, the *Hendrick Hudson* squatted 3.25 feet, using 730 horsepower, with a swell 3.7 feet high.

Observations *c, d, e, f* and *g* were taken at the same point in the river at different stages of the tide. Observations *f* and *g* are particularly interesting. The depths were nearly the same. In observation *f* the steamer was running at what her officers consider her most economical speed for this depth. During observation *g* they were endeavoring to force her to the limit, utilizing enough power to drive her at high speed in deep water, and had reduced her clearance to almost zero. These two observations furnish an example of the great influence of shoal water on power consumption and possible speeds. In view of the apparent discrepancies between observations *d* and *e,* and *f* and *g*, it should be noted that the *Robert Fulton* is a smaller vessel than the *Hendrick Hudson.*

As stated earlier, a tabulated record of the field observations is shown herein as Plate I. This table contains some data on all of the elements entering into the solution of the problem, except a record of the indicated horsepower during each observation. The *Hendrick Hudson* is equipped with a three-cylinder, compound, inclined, direct-connected engine, and the other steamers with low pressure walking-beam engines. All are side-wheel steamers. Several attempts were made to obtain the horsepower consumption by taking indicator diagrams, but, on account of the character of the engines and the great expense involved, it was found impracticable to obtain, by this method, a reliable horsepower record for any considerable number of observations. A record of the number of revolutions per minute and the steam pressure was kept, but the results obtained in computing the power from this data were so diversified and unreliable that they have been rejected. Hence, it has been necessary to compute the power consumption.

From his experiments in the model tank Professor Sadler has platted three speed power curves for speeds between 7 and 21 miles per hour, one curve for a depth of 15 feet, one for a depth of 20

feet, and one for "deep water," meaning water so deep that the effect of the bottom is inappreciable. A study of these curves, in connection with the data obtained in the field, indicates that for power consumption under 1,000 horsepower, curves for depths of 12, 13, 14, 16, 17, 18, and 19 feet can be interpolated with reasonable assurance of accuracy. This gives a set of curves showing within limits the relation between power consumption and

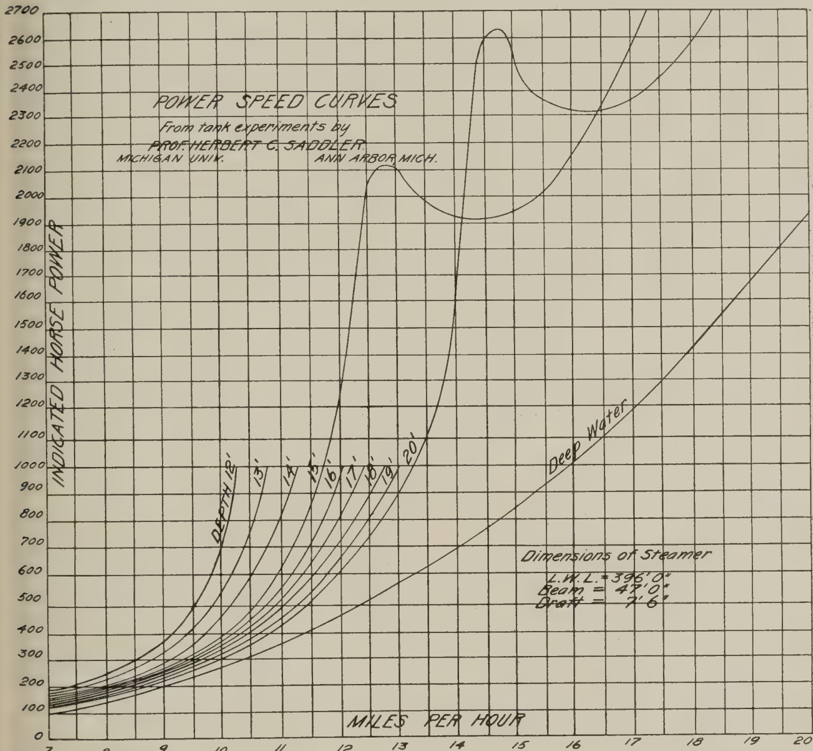


Plate II.

speed for these depths for the model of boat used. These curves are shown on Plate II.

In Professor Sadler's curves, the horsepower was deduced theoretically from measurements of the resistance of his models to towing. The field observations induce the belief that a vessel moving under her own power squats much more than when being towed, and that this increased squat greatly increases her resistance to motion. Hence, Professor Sadler's values for horsepower are believed to be materially lower than the actual power required

in shoal water where the squat is great. It is thought, therefore, that the results of the deductions hereinafter based on Professor Sadler's relations between speed and power are exceedingly conservative, and give values of squat and power actually too low for corresponding depths and speeds.

By a careful analysis of the field data on Plate I, the speed power curves on Plate II, and the limited record of actual power consumption available, the following conclusions were reached:

a. Assuming that for a given depth the indicated horsepowers vary directly as some function of the velocities, and using the equation

$$\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2}=f\frac{(V_1)}{(V_2)}$$

where V_1=the smaller velocity, it was found that for any depth between 12 and 17 feet and for speeds between 7 and 11 miles per hour the equation will have this form

$$\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2}=0.73\frac{(V_1)}{(V_2)}.$$

b. Assuming that for a given depth the squats vary directly as some function of the indicated horsepowers, and using the equation

$$\frac{S_1}{S_2}=f\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2}$$

where $(\text{I. H. P.})_1$=the smaller value for indicated horsepower, it was found that for any depth between 12 and 17 feet and for speeds between 7 and 11 miles per hour the equation will have this form

$$\frac{S_1}{S_2}=1.45\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2}.$$

c. Assuming that for a given velocity the indicated horsepowers vary inversely as some function of the depths, and using the equation

$$\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2}=f\frac{(d_2)}{(d_1)}$$

where d_1=the smaller depth, it was found that for any depth between 12 and 17 feet and for speeds between 7 and 11 miles per hour, the equation will have this form

$$\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2}=0.96\frac{(d_2)}{(d_1)}.$$

d. Combining the equations in *b* and *c* we have

$$\frac{(\text{I. H. P.})_1}{(\text{I. H. P.})_2} = 0.96 \frac{(d_2)}{(d_1)} = 0.69 \frac{(\text{S}_1)}{(\text{S}_2)}$$

$$\frac{\text{S}_1}{\text{S}_2} = 1.39 \frac{(d_2)}{(d_1)}.$$

All of the above relations are approximate and hold only for the speeds, depths, and model indicated. The results of the above deductions are shown in the form of curves on Plates III, IV, and V.

These relations were deduced for and apply exactly to one class and size of steamer only. By using Froude's Laws of Comparison, a similar set of relations may be obtained for other types. However, for use in determining channel requirements in the upper Hudson, it is thought proper to assume that the relations as given above apply to a vessel not over 500 feet long, with beam over guards of 90 feet, beam of hull 60 feet, and a load draft at rest of 10 feet, as the standard steamer to be accommodated. Under these assumptions the following deductions can be made:

1. That in 12 feet of water safe clearance can not be maintained.
2. That in 13 feet of water safe clearance can be maintained with a speed less than 8 miles per hour.
3. That in 14 feet safe clearance can be maintained with a speed of 10½ miles per hour, but that at this speed the power consumption will be uneconomical and the swells damaging.
4. That in 16 feet a speed of 12 miles per hour can be maintained safely and economically.
5. That speeds greater than 14 miles per hour are uneconomical or unattainable, except in very deep water.

The large steamers of the Day Line are designed to operate economically at a speed of 20 miles per hour in deep water. They are scheduled to make the run from New York to Albany as a daylight trip in about ten hours, and the volume of their through business depends largely on the time element. At present, the average speed of both day and night boats north of Hudson is from 7 to 12 miles per hour. The night boats are designed for an economical speed of 15 miles per hour. As they make no stops between Albany and New York, the question of speed is not so important to them as to the Day Line. The most important benefits

to be gained by an increase in depth would be—for the day boats, greater speed, and, for both day and night boats, economy in power and a safe clearance.

With respect to freight traffic, two elements are of particular importance: the draft of tow-boats, and resistance to motion of barges, either self-propelled or under tow. The volume of the prospective traffic to and from the barge canals has been estimated at fifteen million tons per annum. The local tonnage and that transferred by rail amounted to about three million seven hundred thousand tons in 1910. Making due allowance for the business of passenger steamers, a very conservative rate of increase would place the probable total barge traffic cn the river at about twenty million tons per annum. This would require twenty thousand cargoes per annum, if the average cargo is one thousand tons.

Experience has shown that from 13 to 15 feet is the most economical draft for tow-boats of the type used on the lower Hudson and in New York Harbor. As noted above, shallower draft boats are now in use on the upper Hudson for heavy work, because only of lack of channel depth now and in the past. At slow speed when pulling, the propeller tugs do not squat appreciably. Hence, with 16 feet of water, a 14-foot tug can maintain a 2-foot clearance, while to a far greater degree than in the case of passenger or freight carriers the slow moving, relatively small, stoutly built tug can operate on less clearance with comparative safety. Based on the best boats now in service on the Hudscn, the following seems to be the limiting indicated horsepowers practicable for various draft tugs:

Draft in Feet.	*I. H. P.*
6-8	150-300
8.5-10	500-800
13-15	1,000-1,800

With equal indicated horsepowers, the pulling power generally increases with the draft. The smallest tugs in service are operated with a crew of nine men, and the largest with a crew of twelve men. For a given total pulling power, however, the coal consumption varies more nearly with the total horsepower required, however distributed between tugs. The cost of wages and repairs is more nearly a function of the number of boats. These figures give some idea of the economy in using deep draft boats.

It has been found, by experiments and experience, that the standard formulæ for resistance to motion in general use in marine

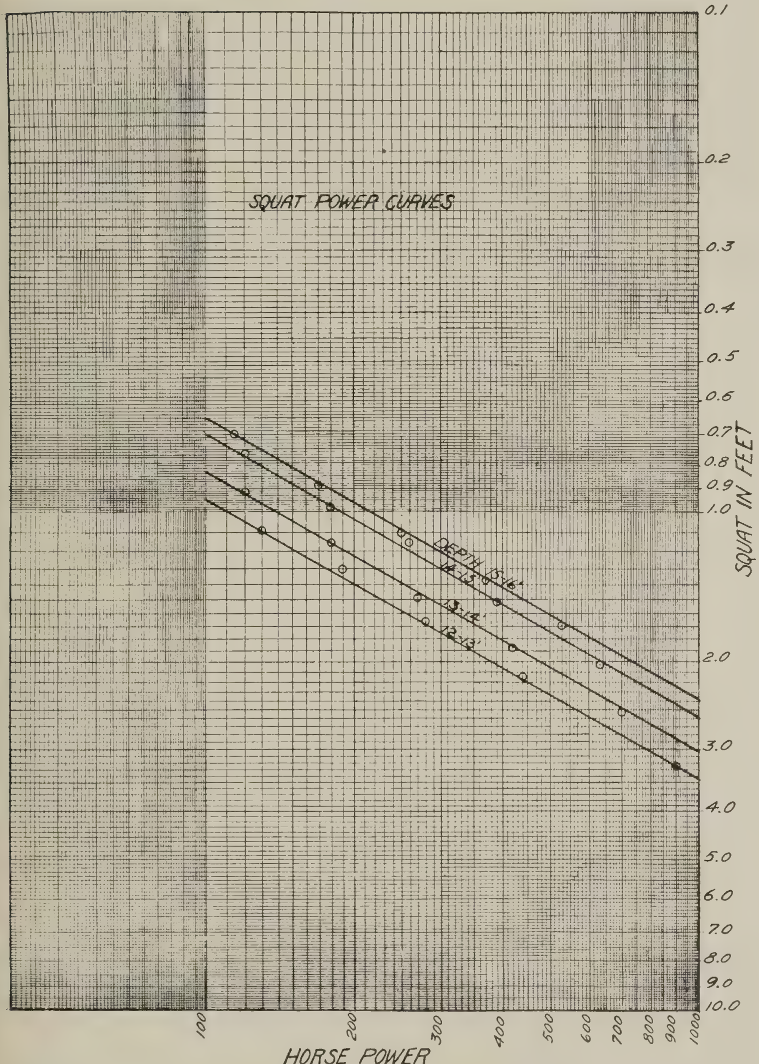

SQUAT POWER CURVES
DEPTH 15'-16'
14'-15'
13'-14'
12'-13'
SQUAT IN FEET
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
100
200
300
400
500
600
700
800
900
1000
HORSE POWER

Plate III.

architecture do not give reliable results for the channel conditions prevailing on the upper Hudson. The best available guide is believed to be contained in curves deduced by Professor Sadler as a result of his model tank experiments referred to above, and are presented in an article entitled "The Resistance of Some Merchant Ship Types in Shallow Water," read at the Eighteenth General Meeting of the Society of Naval Architects and Marine Engineers, on November 16 and 17, 1911. Based on his curves for "full type of cargo boat," the following table of resistances and effective horsepowers was computed for a barge 270 feet long and 40 feet beam, drawing 9.5 feet. The effective horsepower is that computed as necessary to exert the towline resistances given.

Towline Resistances.

Speed.		Towline tension in tons.*			Effective H. P. required to overcome towline tension=½ I. H. P.		
Miles per hour.	Feet per second.*	D.=12′.	D.=14′.	D.=16′.	D.=12′.	D.=14′.	D.=16′.
4	5.866	1.35	0.75	0.66	28.8	25.7	23.5
6	8.8	2.1	1.5	1.35	67.5	48.0	43.0
8	11.74	5.25	3.0	2.55	251.4	128.1	108.86

Records of actual coal consumption over an extended period show that a modern tug belonging to the Standard Oil Company used, when pulling at an average speed of 6 miles per hour in deep water, 250 pounds of coal per mile, or 1,500 pounds per hour. Assuming that she was utilizing her full rated power and that the effective horsepower exerted on the tow-line is one-half the indicated horsepower, and using the figures for tractive resistance and effective horsepowers given in the above table, we then get the following figures for the coal consumed by this boat in pulling one hypothetical barge 1 mile with different depths of water.

Coal Consumption in Tons (2,000 Pounds).

Speed: Miles per hour.	Depth in feet—		
	12	14	16
4	0.0216	0.0189	0.0176
6	0.0337	0.0240	0.0215
8	0.0943	0.0480	0.0408

*Compiled by Professor Sadler.

In the 38 miles between Troy and Hudson, there are about 8 miles of channel with a natural depth greater than 12 feet at lowest low water, or 30 or more miles of channel to be dredged under the existing project. With a 12-foot depth, the coal consumption at 4 miles per hour would be about 0.648 ton, and, with 14-foot depth, 0.568 ton, while at 6 miles per hour, it would be 1.012 and 0.720 tons for these two depths, respectively, effecting a saving of from 0.1 to 0.3 of a ton for each trip with a single barge

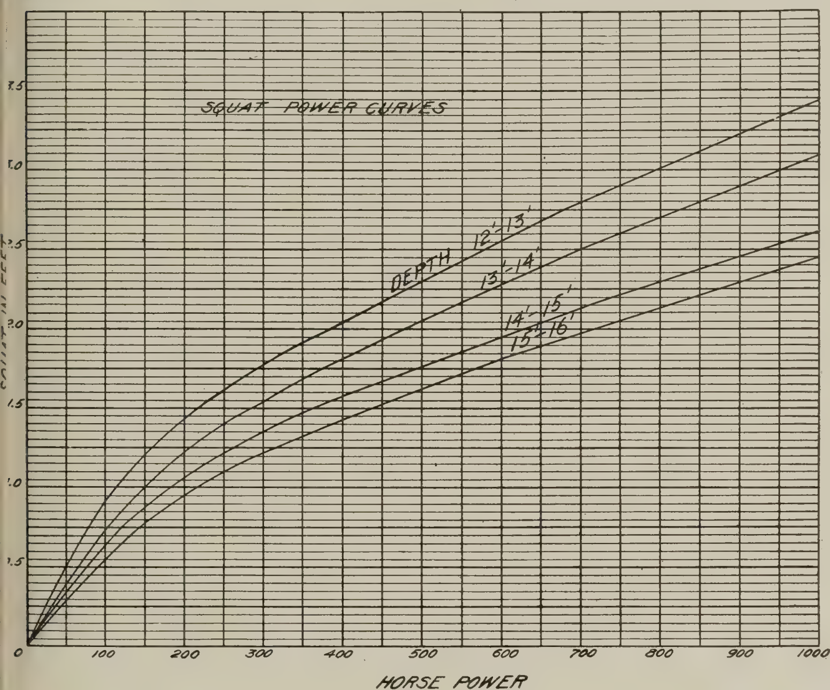


Plate IV.

between Troy and Hudson with a 14-foot channel. It will be noted that the above deduction makes no allowance for loss in efficiency in the tug when operating in shoal water, a loss which is not inconsiderable, and affects the utilization of only a small part of her power. In a stationary dynamometer test under favorable conditions, one of the Hudson River tugs lately pulled 7,600 pounds, developing 176 I. H. P. Taking the effective horsepower exerted on the tow-line as one-half the indicated horsepower, her maximum speed pulling 7,600 pounds would then be 6.37 feet per

second, or, 4.34 miles per hour, from the following equation:

$$\text{E. H. P.} = \frac{\text{I. H. P.}}{2} = 88 = \frac{7{,}600\text{V}}{550}.$$

V=speed in feet per second. Her tow-line pull would then be 3.8 tons (of 2,000 pounds) at 4 miles per hour, 2.75 tons at 6 miles per hour, and 2.061 tons at 8 miles per hour. Referring to the table given above, she could then pull in 12 feet of water:

3 barges at 4 miles per hour;
1.8 barges at 6 miles per hour;
1 barge at 8 miles per hour.

In 14 feet of water:

5 barges at 4 miles per hour;
2.5 barges at 6 miles per hour;
1.2 barges at 8 miles per hour.

In 16 feet of water:

6 barges at 4 miles per hour;
3 barges at 6 miles per hour;
1.5 barges at 8 miles per hour.

Or with three barges, the boat could save about two hours in the run between Troy and Hudson if she had 14 feet of water instead of 12 feet, or, at a speed of 4 miles per hour, could haul nearly twice as many barges. The coal consumption of this tug when pulling at full power is between 1,500 and 2,000 pounds per hour. Hence, in hauling three barges from Troy to Hudson, she would save about 2 tons of coal. Hence, if this tug carried 15,000,000 tons per annum between Troy and Hudson, three barge loads averaging 1,000 tons each at a time, her annual saving in coal would be about 10,000 tons, or, at \$3.25 per ton, \$32,500 per annum. If a speed of 4 miles per hour is maintained, her coal consumption per mile would be about 350 pounds. With 12 feet of water her maximum load is three barges; with 14 feet, five barges. The coal consumption per barge would then be about 3,500 pounds for the run with 12 feet of water, or 2,100 pounds for 14 feet of water. Thus, the deeper water effects a saving of about \$2.26 per barge. With 15,000 barge loads, the annual saving in coal would be \$33,900.

The above deductions are based on assumptions which are not fully substantiated and formulæ which are not strictly applicable to the conditions. However, the results are consistent with actual practice as far as ascertainable. From the records of the

largest towing company on the river, it appears that the average coal consumption between Albany and Hudson is 33 per cent of the total between Albany and New York, while the distance from Albany to Hudson is about 30 miles, and from there to New York about 120 miles. The towing time between Hudson and New York is from twenty-four to thirty hours, and between Albany

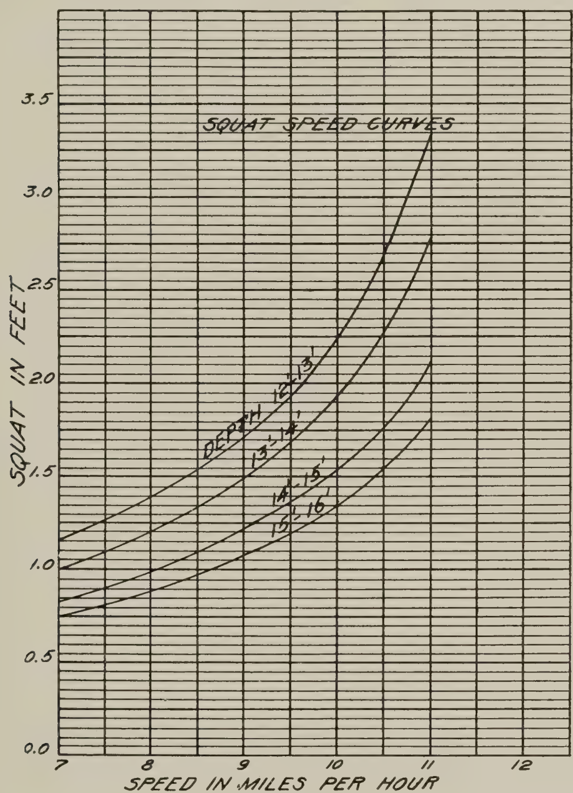


Plate V.

and Hudson from seven to twelve hours. During high water the tug *Pocahontas* (one of the most powerful in service, with normal draft of 13 feet) with a load of about twenty-five canal boats, takes about four hours to run 14 miles from Albany to New Baltimore, with a depth of from 12 to 14 feet, and two hours to run from there to Four Mile Point (12 miles) with a depth of from 15 to 17 feet, but including a few bad shoals. The tug *Murray*, of about 300 I. H. P., made, during the last year, 107 round trips

between Albany and New York, towing one large barge drawing 9 feet. From his actual observations, her engineer computes that a difference in depth of 2 feet, resulting from tide or freshet, effects an average saving of one-half ton of coal between Albany and New Baltimore.

River and Harbor Notes from Foreign Lands

COMPILED BY

Lieut. F. B. DOWNING
Corps of Engineers

COLOMBO HARBOR, CEYLON.*

The recent improvement of the immense artificial harbor of Colombo, Ceylon, emphasizes the fact that it ranks as the sixth or seventh most important harbor of the world, and as the third most important port of the British Empire, of which the first is London and the second Hong-Kong.

In 1871, two years after the opening of the Suez Canal, the shipping entering Colombo was practically nothing. In that year the British Admiralty decided that Colombo should be the port of Ceylon, and accordingly a breakwater protection was planned to shelter the roadstead. This project was formally begun in 1875 under the direction of its author, Sir John Coode. It included an area of 500 acres between the southwest breakwater and a similar work west of the present northeast breakwater. In 1877 the first structure was partially completed, and the immediate benefit was evidenced by the increase of the harbor tonnage to 600,000 tons. Experience demonstrated that the original project would not accommodate future trade, and the project was enlarged to that shown in the accompanying plan (p. 615) of the completed works.

The importance of Colombo is due to its geographic situation on the great trade routes of the Indian Ocean, which have been developed by the traffic of Europe with Africa, Australia, India, and the East, through the Suez Canal.

The shipping frequenting the port of Colombo for 1911 was 9,000,000 tons, exclusive of coasting trade. In 1908, the entire American fleet on its world cruise remained several days in the harbor without the slightest inconvenience to the thirty or forty mail liners that shipped during the same time.

The port affords a sheltered anchorage of 660 acres, which is secured by a length of breakwater of 10,000 feet. It is provided

Engineering, London, Vol. XCIII, No. 2422, pages 730, 731.

with ample wharves, a graving dock, repair ways, warehouses, etc., and coaling facilities with storage for one quarter of a million tons of coal, and all of the accessories necessary to a large mercantile and naval base of supplies.

The completion of this work is especially noteworthy. It was carried out by the engineers without the intervention of contractors; it was built as an entire project, the continuous work of thirty-seven years, looking to the completion of a definite plan of improvement, which was executed in restraint of severe elemental forces, at great risk of damage to plant and and partially completed works from the monsoons blowing six months in the year. It was accomplished within the estimates; its cost was about $15,000,000.00.

The southwest breakwater, as shown on the plan, extends from the western end of the harbor foreshore in a northerly direction for about 3,500 feet, it then curves eastward and terminates in a head. The length of this breakwater is 4,210 feet. A light-house is built into the head. The original project planned to complete the enclosure by a second breakwater, to be built from the land and extend toward the southwest breakwater, with a single entrance to the harbor between the two arms. However, to enlarge the harbor to accommodate prospective trade, it was decided to replace this northern arm by an arm running between Mutwal Point and the north end of the arm already built (the southwest breakwater). This arm consists of two parts, an island breakwater and a shore arm. The latter, called the northeast breakwater, is 1,100 feet long and built of rubble. The space between it and the southwest breakwaters is partially closed by an island breakwater 2,656 feet long. Two entrances to the harbor, 700 feet and 800 feet wide, respectively, are left around the ends of the island breakwater. The object of these entrances is two-fold, facilitating the movement of ships passing into and out of the harbor, and also for the sanitary purpose of inducing currents in the enclosure where the tidal range is only 2 feet.

It was found that gales from certain directions produced a swell sufficient to inconvenience the use of the coal wharves, and in order to prevent this it was decided to protect the southern entrance to the harbor by an additional arm, as shown on the plan. This arm is built in prolongation of the southern breakwater, to which it is joined, and extends 1,800 feet across the front of the protected entrance. Both of the angles at the junction between

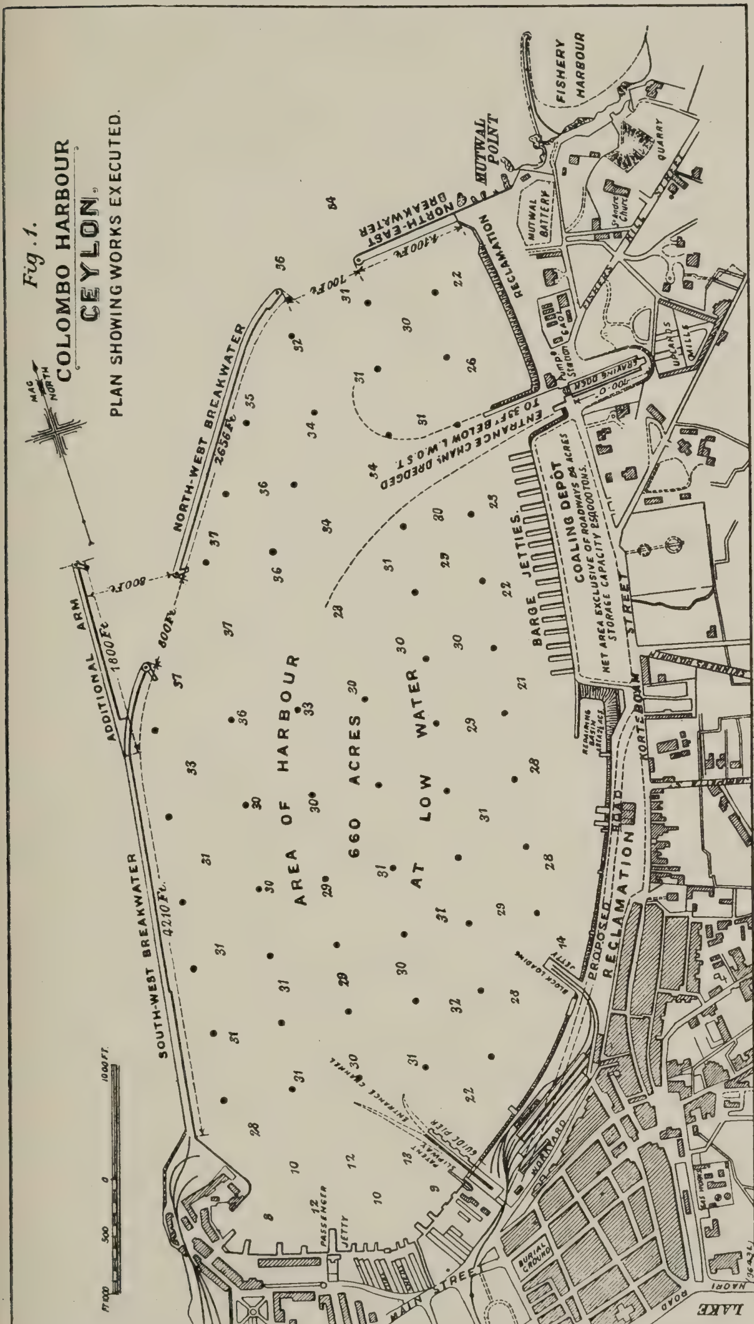


Fig. 1. COLOMBO HARBOUR CEYLON. PLAN SHOWING WORKS EXECUTED.

these structures are filled with "pell mell" concrete blocks to break the force of the waves.

The additional arm, the southwest and the northwest breakwaters are similarly constructed of heavy concrete blocks on a rubble mound. The granite rubble foundation was deposited from barges on to the sea bed. As a rule, one year elapsed before the concrete superstructure was begun. Divers dressed the top of the mound with small rubble. The inner side of the foundation is battered 4 to 1 at the top, 1¼ to 1 below—the seaward side, 6 to 1 at top, 2½ to 1 below. The concrete blocks of the new arm are founded on the rubble, 30 feet below low water. The width at this level is 38 feet, above this the width is 36 feet to the capping which is concrete in mass. The concrete blocks weigh about 30 tons. They were hauled in wagons from the yard to the work, where they were set with a Titan crane, and were accurately placed by divers. They were laid as headers in sloping courses with the joints breaking. The binding was strengthened by concrete in bags, deposited in grooves between the courses. The block work was raised to 8 feet above low water and there finished with a capping of mass concrete, 3 feet 6 inches thick.

The foundations of the breakwaters are protected by 30-ton wave-breaker blocks, deposited "pierre perdue." An apron of concrete in bags protects the heads of the structures.

The additional arm at the seaward end terminates in a head 60 feet in diameter, built of heavy concrete blocks in horizontal courses. The outer blocks in the various courses are doweled together with heavy rail clamps. In addition, there are circular concrete joggles.

Light-houses are built on each of the breakwater heads. The one at the head of the additional arm is built of concrete in mass. The height of the tower is 36 feet 8 inches, the focal plane being about 58 feet above low water. The tower is 23 feet 3 inches in diameter at the base, and tapers to 17 feet near the top. The diameter of the lantern platform is 22 feet. There is fitted a second order, fixed, red dioptric light of 4,800 candlepower.

There are three floors within the tower, 11 feet in diameter and 11 feet apart. They are carried on steel joists. Communication is provided by a circular iron stairway. Oil is stored in tanks built into the concrete work at the ground floor level.

A landing jetty with boat steps and a landing stage on the landward side is built for use in connection with light-house work.

The northeast breakwater, 1,100 feet long, is a rubble mound carried to 6 feet above the high water of ordinary spring tides.

Several important harbor adjuncts were included in the improvement. Perhaps the most important is the dry dock; this is situated at the north end of the coaling depot, at the other end of which a repairing basin is built for coal vessels.

The dock is 700 feet long on the floor. The width of the entrance at the cope-level is 85 feet, the width on the floor is 63 feet. The depth over the sill at low water is 30 feet—the floor is lower at the entrance end than at the head—rising 1 foot in the length of the dock. The sides have a batter of 1 in 12.

A patent slip for hull repairs has also been provided. It is situated at the southern end of the harbor, and will accommodate vessels up to 1,200 tons.

A large area of land around the shore from this shipway to Mutwal Point has been reclaimed. It is probable that the admiralty will utilize a portion of this land north of the dry-dock for a coal storing station. A part of it is already occupied by eighteen piers for coal barges and a coaling depot covering 24 acres, with a capacity of 250,000 tons of coal. The passenger wharves for small vessels and steamers are shown west of the hull-repairing shipway.

The depth to which the harbor has been dredged has been determined by reference to the drafts of vessels using the Suez Canal. The deepening of three quarters of the harbor to 36 feet, so as to accommodate the largest ships that the present depth of the canal will pass, is contemplated. Further dredging will be done if necessary.

Messrs. Coode, Son, and Matthews were the consulting engineers for the work, upon which Mr. John Kyle, Sr., M.I.C.E., Mr. J. Bostock, M.I.C.E., Mr. John Kyle, Jr., and Mr. A. D. Prouse, M.I.C.E., were successively engaged as resident engineers. The Governor of Ceylon, at the ceremony in commemoration of the completion of the work, bore testimony to the "ingenuity, bravery, skill, watchfulness and care" displayed by the engineers and their staff in the conduct of the work.

VALPARAISO HARBOR.*

The Chilian government has awarded a contract for the improvement of Valparaiso Harbor to S. Pearson & Son, Limited, for 40,978,460 Chilian dollars (about $14,752,250 U. S. currency).

*_The Engineer_, London, Vol. CXIII, No. 2943, pages 540, 541.

The work contemplated is a compromise between several elaborate plans of improvement. The breakwater planned by Mr. Kraus was modified on account of its great cost, due to the excessive depth of water. On the other hand, no provision is made for the disposal of detritus brought down in the numerous creeks and culverts by the winter rains.

The adopted project calls for the following:

1. A breakwater, 945 feet long, at Puerta Duprat.

2. A wharf, 2,066 feet long, between the fiscal wharf and Puerta Duprat, and the filling behind the wharf wall for the reclamation of land for warehouses, railway tracks, etc.

3. Strengthening and lengthening the fiscal wharf to a length of 1,214 feet.

4. A wharf, 689 feet long, between the south end of the fiscal wharf and the new mooring jetty.

5. A mooring jetty, 820 feet long and 328 feet wide, with quay walls on each side, including the necessary filling.

6. A coal pier, 656 feet long and 98 feet wide, with coal hoists, cranes, railway tracks, etc.

7. Warehouses, customs buildings, light-houses, offices, etc.

The general plan of the proposed improvement is shown in the illustration (p. 619) from the *Engineer* (May 24, 1912).

The breakwater, ABC, begins at Puerta Duprat, and runs E.N.E. to a depth of about 150 feet at mean tide. A second breakwater, CD, is contemplated to be built in extension of the first. It, however, will not be built unless experience proves it necessary. The wharf (2) provides a frontage of 2,066 feet in a depth of about 60 feet at mean tide; the mooring jetty provides a wharfage length of 1,970 feet; the fiscal wharf, and the wharf between it and the mooring jetty, provide a frontage of about 1,900 feet. The total length of frontage thus provided (where steamers may lie in quiet water at all seasons) is about 6,000 feet, exclusive of the coal pier. Between the mooring jetty and the coal pier, the foreshore will be widened to 130 feet. It will be protected by a stone breakwater.

Taking into account the shipping facilities of the new port of San Antonio, about 50 miles south of Valparaiso, and the increased traffic due to the opening of the Panama Canal, it is estimated that the completed improvements of Valparaiso will make it rank as one of the best harbors on the Pacific Coast. It will accommodate the annual entry of 1,850 vessels, and the handling of 3,700,000 tons of cargo. It will provide a protected anchorage of 220 acres,

more than 6,000 feet of wharf frontage, and the storage room provided by four bonded warehouses, with an area of 32,300 square feet, four stories each.

The work required is estimated as follows:

Dredging in sea bottom	677,270 cubic yards.
Filling for reclaimed areas	2,750,669 cubic yards.
Granite rubble—breakwaters, foundations, etc.	1,603,656 cubic yards.
Concrete in breakwaters, wharf walls, etc.	425,366 cubic yards.
Steel and iron work	13,600 tons.
Railway and frame lines	77,080 feet.
Paving roads, etc.	195,000 square yards.

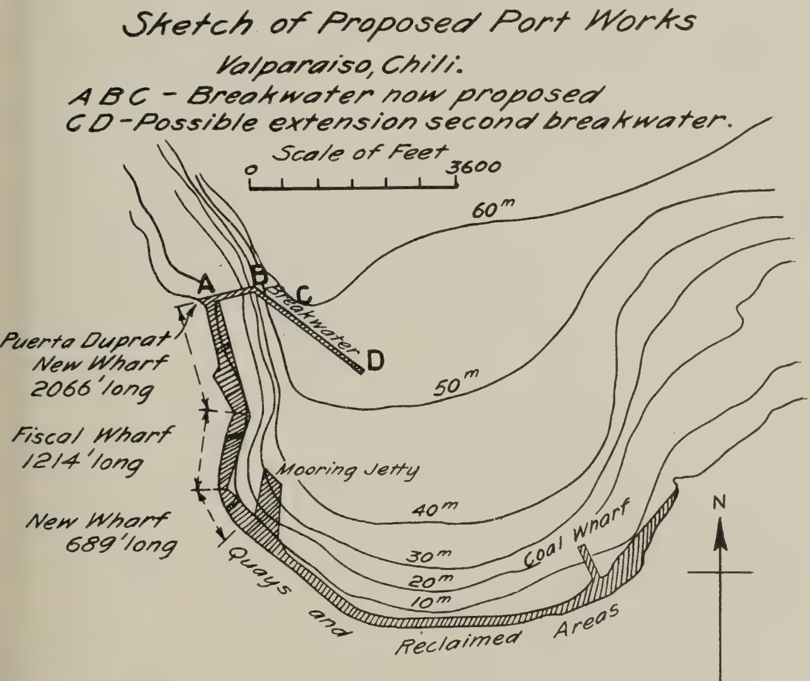


Fig. 2.

IMPROVEMENTS TO THE PORT OF ANTWERP.*

A government commission was appointed in 1907 to examine into and report upon the suggested improvements to the port of Antwerp and its approach from the sea. Two principal schemes of improvement were considered by the commission. Its report was submitted late in 1911, after extended consideration of several schemes, and it recommends a compromise between the two principal ideas considered. It is understood that the report has been

*The Engineer, London, Vol. CXIII, No. 2941, pages 481, 482.

adopted by the government which, with the city of Antwerp, will undertake the improvements recommended.

The first of two schemes, which is shown (fig. 3, p. 621), proposed an entirely new channel for the river, which would start from near the entrance to the docks of Antwerp and extend in a wide curve (a distance of 6 miles) to Kruisschans, at the mouth of the river. The minimum width of this canal was to be 450 feet. The half length at the Antwerp end was to have a radius of about 26,000 feet, the remainder of the channel a radius of about 40,000 feet. The concave bank of the channel was to be built entirely of masonry walls; the convex bank, of earth revetted with stone. Two locks were proposed, one at Kruisschans, the other about midway between it and Antwerp. The wharves, which were to have a frontage of 24,000 feet, were to be built on the concave bank with a depth of water from 29 to 36 feet. It was intended that the canal would supplement, but not entirely do away with, the old river channel.

The project recommended by commission is shown (fig. 4, p. 623). It proposes regulation of the old channel by realignment of the banks, and the building of a lock at Kruisschans, which would open into a 400-foot channel that would lead through a series of locks to the docks and river at Austruweel, just below Antwerp. Along the concave bank at Austruweel a length of about 14,000 feet of wharf is to be built. Other wharves are planned along the concavity above and below the lock at Kruisschans. Thus there would be provided a total wharf frontage of about 26,000 feet. The scheme also presents other possible wharf locations, including the left river bank, which would give a total wharfage of about 56,000 feet.

Dry excavation is planned for the removal of spoil wherever it is practicable, but much of the work must be done by dredges. The work on both banks is to be carried out simultaneously, and the current in the new channel being led by dikes, or groynes, will be utilized to assist in the work of regulation. These dikes will hasten the silting up of the abandoned channel.

The main entrance lock to the docks is to be 1,312 feet long and 147.5 feet wide, with a minimum depth of 40.3 feet at low tide. The city of Antwerp is to construct the docks, whereas the government will build the main lock and carry out the river improvement. The entire work is estimated to cost about $8,500,000.00.

The principal arguments that were advanced in the advocacy of

the first plan of improvement were that the duration of flood tide at Austruweel would be lengthened about 30 minutes; that easy access would be afforded to the lower reaches of the river, and that the proposed scheme was the simplest possible solution of the problem.

The opinions urged against this plan were: That great uncertainty existed as to which channel, the old river bed or the new canal would silt up; that probably the channel of the new canal

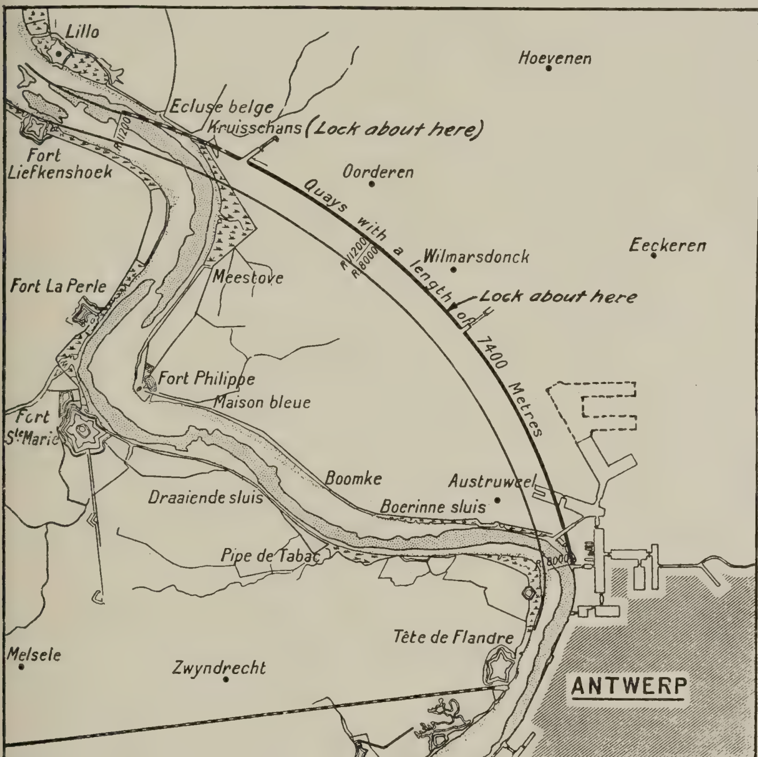


Fig. 3. Map showing proposed "Grand Cut" considered by the Commission.

would be unstable, since in natural streams the greater depths occur at the sharpest curves, and reverse curves give the most stable channels; that the great continuous length of wharves would further complicate maintenance; and that the revised project, in attempting to ameliorate the existing channel by regulation was the more rational; that it was also cheaper, and could be completed in a shorter time.

ENLARGEMENT OF THE KAISER WILHELM CANAL.*

When the Kaiser Wilhelm Canal, from Kiel, on the Baltic Sea,

*Engineering, London, Vol. XCIII, No. 2425, page 844.

to Brunsbüttel, on the lower Elbe, was opened in 1895, it was foreseen that ultimately it would have to be widened and deepened. But probably no one would then have suggested the dimensions of the canal and locks that are now conceded to be indispensable. The recent development of shipping and of naval architecture have exceeded all expectation.

On the other hand, however, had the present dimensions been originally planned, very probably the canal would not have been built. Some undertakings are first projected on a grand scale, in order to obviate future extensions; others are instituted as modest beginnings which are improved as the needs arise. By the latter method, perhaps the first cost of the enterprise is greater than by the former method. But there is thus avoided the payment of heavy interest on a part of the capital invested in a work greatly superior to present needs. The policy of the extension system of work seems on the whole to be one of sound business.

The new Kiel Canal in some respects surpasses the Panama Canal in magnitude. The two locks at the entrances are to be replaced by the largest locks in the world; the enlarged canal will contain twice as much water as at present, both the width and depth are to be increased; bridges are to be rebuilt and sharp curves eliminated in the improved canal, and all this work is to be done without interrupting traffic. A sum of 223,000,000 marks (more than $50,000,000.00) has been appropriated for the work. Operations were begun in 1909 and are to be completed in 1915.

The bottom width of the canal is to be enlarged to 144 feet; the depth is to be increased to 36 feet at ordinary canal level, giving a water level width of 335 feet and a section of 8,880 square feet, viz, double the original section. The minimum radius of curvature is to be 5,900 feet, and most of the curves will have a radius of more than 10,000 feet. Ten passing places will be provided with lengths of from 2,000 feet to 4,600 feet, with bottom widths of 437 and 292 feet, the latter width when ships can pass on both sides of the mid-water line. More than thirteen hundred million cubic yards of soil will be excavated, two-fifths of which are above water level.

The usable dimensions of the locks are: Length, 1,083 feet; width, 147.6 feet, and depth, 45.2 feet, to provide for possible future deepening of the canal; the corresponding dimensions of the Panama locks are: 1,000 feet, 110 feet, and 40 feet. Each lock is to be fitted with three electrically-operated sliding gates weighing 1,000 tons each. The middle gate is to serve for dividing the

lock chamber into two lengths of 328 feet and 725 feet, respectively —frequently sufficient for the smaller ships.

The enlargement of the old locks without the interruption of traffic is out of the question. Completely new locks are accordingly being built next to the old ones. Much of the excavation in the dry has been completed; a stone revetted dam being left along the present canal bank, so that the dry excavation could be carried

Fig. 4. Map showing proposed alterations to the new channel recommended by the Commission.

further. For this it was necessary to lower the ground water level. This was done by electrically driven centrifugal pumps, running day and night, using about 600 horsepower in pumping from the bottom of the pipe wells 65 feet below the canal level. In connection with this work, where the foundations make it necessary, piles are driven into the ground upon which the foundation of the new walls are supported. Most of the excavation is being done by contract.

In some localities there is no room for moving materials on inclined planes. Cable cranes with vertical lifts are used. They, together with the concreting plant and other machinery, are electrically driven from a central power station.

The labor question has been well handled. Last year as many as 7,600 men were employed. Preference is always given to German labor; and though it has been necessary to employ foreigners, the contractors are required to satisfy the administration that German workmen can not be secured. Every man is medically examined before he is accepted, and he remains under medical control. The unmarried men usually are required to live in barracks, under the supervision of the administration. The charge for lodging there and three meals is about 25 cents a day. Every effort is made to suppress alcoholic indulgence, and to conserve the health of the men. Sunday work and overtime are not permitted, except by special authority granted for exceptional circumstances.

CONCRETE STEEL BARGE FOR THE MANCHESTER SHIP CANAL.

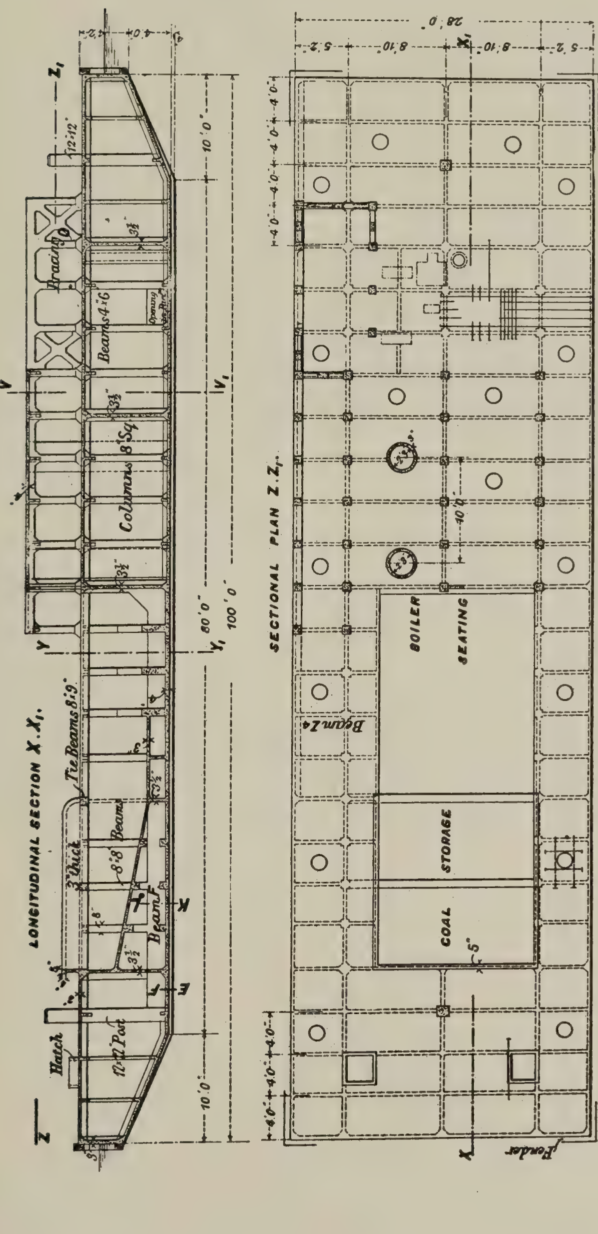
As far back as 1849 a reinforced cement boat was built by M. Lambot-Miravel, and exhibited at the French Universal Exposition of 1855. Since 1906, a reinforced concrete dredge has been regularly operated in France.

Other such boats and barges have recently been built in Germany, Italy, and Panama.

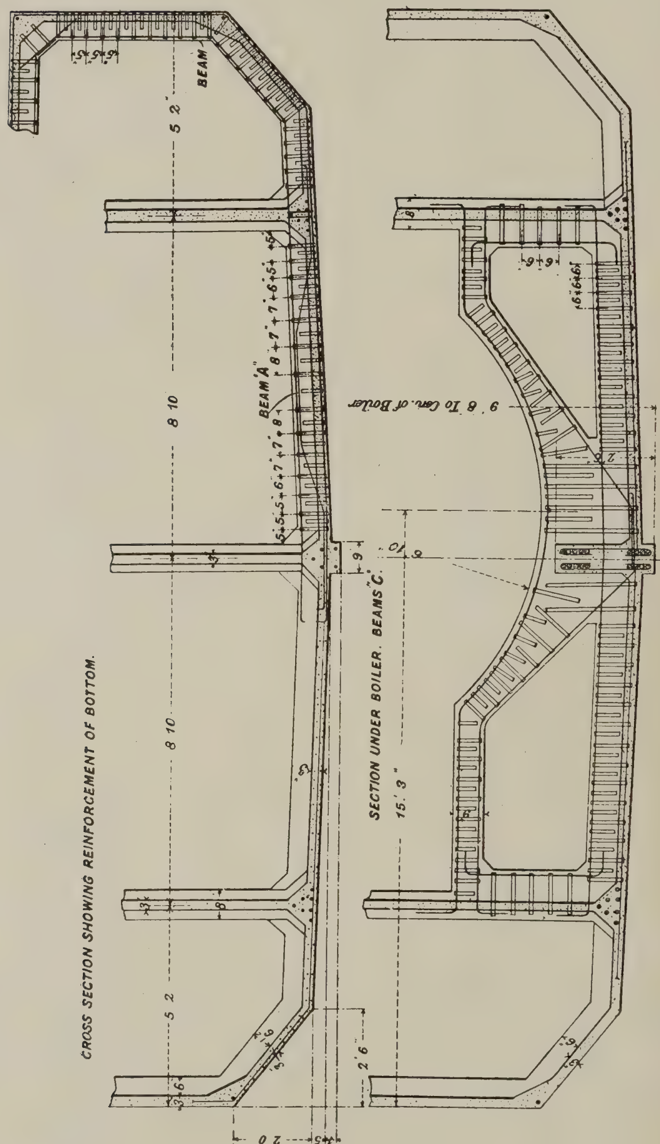
A ferro-concrete barge has just been completed for use as a sludge pumper in the Manchester ship canal by Messrs. L. G. Mouchel & Partners, London. This boat is described in *Engineering* of June 14, 1912, from which the following description and illustrations are taken.

The ponton measures 100 feet long by 28 feet wide by 8 feet 6 inches deep. It will draw about 6 feet 6 inches loaded. It is divided into compartments by three longitudinal and four transverse water-tight bulkheads. The boiler space and coal bunker compartments are separately framed. A main deck extends entirely around the machinery and coal spaces, and an upper deck is built over one half of the boat, a passageway on the main deck being left around one side and the end of the upper structure. Steel towing and snubbing posts are provided.

The compartments aft of the coal bunkers are entered through hatches from the main deck. They are used for the storage of ropes and other materials. The coal bunkers provide a capacity



Figs. 5 and 6. Plan and section of concrete steel barge for the Manchester Ship Canal.



Figs. 7 and 8. Cross sections showing the reinforcement of the bottom and under the boiler.

of 40 tons. They are floored with 1/4-inch steel plates. The adjacent forward spaces are built for the installation of the machinery of the boat, which consists of a 58-ton marine boiler, a compound vertical steam engine and condenser, three centrifugal pumps, and three steam winches. The other forward compartments are used to trim the vessel under various loads by easy control of the amount of water carried therein. The machinery spaces are shown uncovered in the illustration, but are to be roofed over in some way not described.

This new boat is to form part of the regular dredging fleet of the

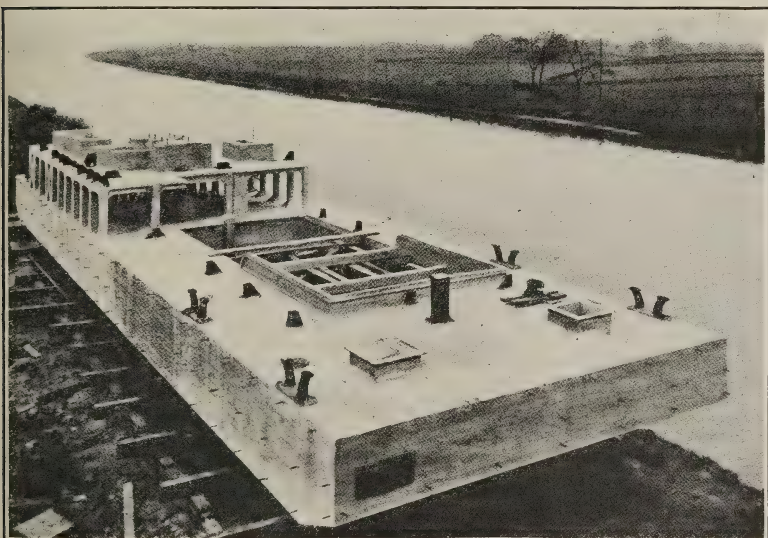


Fig. 9. Concrete steel barge for use on Manchester Ship Canal.

Manchester ship canal, and is to be used in the disposal of dredged materials along low-lying banks, improving the quality of the land and conducing to the economical disposal of the spoil. It will be towed to the selected site and there moored. The loaded barges will then be made fast alongside. The suction and delivery pipes will be placed as required and the barges unloaded by pumping their contents to the adjacent land.

The reasons stated as determining the selection of reinforced concrete instead of steel for this boat were its lower initial cost, the elimination of maintenance charges and the readiness with which repairs could be executed in the event of collisions.

Water Supply of the Orleans Canal (France) by the Elevation of Water from Pool to Pool*

BY

M. ROUSSEAU
Engineer-in-Chief of the
"Ponts et Chaussees"

On the Bourgogne Canal, at each of the three locks nearest the Saône, pumping apparatus has been installed for the elevation of water from the lower to the upper pool. The water supply of the three pools is maintained by these pumps, which raise the river water into them, either directly, in the case of the first pool, or by successive steps, in the case of the higher pools. The pumps are operated by current from a hydroelectric plant established for the purpose on the bank of the Saône at the Saint-Jean-de-Losne dam, where the fall furnishes the necessary power. The Lens Canal also has, at two of its locks, pumps for the supply of the pools above, the pumps receiving their power from an electric plant belonging to a private mining company.

The same method of water supply is to be applied to the whole of the Orleans Canal. There will be especially constructed for this project a steam power plant, which will generate electric current for distribution to the eleven locks on the slope toward the Loire, covering a total distance of 16.8 miles. The object of this article is to set forth the reasons for the adoption of this plan and to describe the installation.

ADVANTAGES OF THE NEW METHOD OF WATER SUPPLY.

The Orleans Canal has heretofore received its water supply from (first) a system of thirteen reservoirs, having a total capacity of 152,000,000 cubic feet; (second) eight small creeks, which discharge into the canal, and (third) numerous drains. The total available supply from these sources is insufficient during an average period of one hundred days per annum. For this length of time navigation must be wholly suspended, or, at the best, a depth of less than 1 meter is available. This is one of the causes of the lightness of the traffic, which never exceeds 50,000 tons per annum on the entire canal.

It is recognized that the only available remedy for this state of

*Translation by Lieut. Carey H. Brown, Corps of Engineers, from *Annales des Ponts et Chaussees*, January-February, 1912.

affairs is to supply the water from the Loire. Calculations have shown that, during periods of great drought, it will be necessary to obtain from the river a volume of 185 gallons per second. This amount is intended not only to supply the losses due to evaporation and to filtration into the soil, but also to help supply the growing needs of traffic. For it now seems reasonable to suppose that the traffic will, in the future, reach a maximum of 300,000 tons as a result of the improvement of the existing waterway and of its prolongation to the city of Orleans, which is now in course of construction.

The general direction of the canal is but slightly inclined to that of the river, and the cross-country distance from the river to the summit level is therefore not great. As a result, the solution which first presented itself contemplated the establishment of an hydraulic power plant at Châteauneuf-sur-Loire, the point on the river nearest the summit of the canal. But, on account of the small slope of the Loire, it would be necessary to construct a channel, leading from a point 2.5 miles upstream, in order to secure sufficient water power, and as this supply channel would pass through a valley subject to inundation it would be necessary to construct it entirely of masonry. Beginning at the power plant, the water was to be raised to a height of 21 meters through a pressure conduit 1100 meters long. From this conduit the water would flow through a feeder channel 9.3 miles long, passing underground for a distance of over .75 mile. This project was abandoned as too costly.

The installation of a steam power plant at the same point was next considered. All the expense of the masonry flume would thereby be avoided; the pressure conduit and the feeder channel remained the same. On the other hand, the power plant proper would eventually be more costly, due to the continual expense for fuel, which would increase with the traffic. Thus a considerable saving in construction cost over the preceding project would be overbalanced and the steam plant would in the end be the more expensive of the two.

Dissatisfaction with these projects brought about a search for an entirely new one. In obtaining a water supply from the river at Châteauneuf, as planned, the entire amount of water required by the canal must be raised to the summit level. If, on the other hand, the water should be raised from pool to pool, beginning at the lock at Combleux where the canal enters the river, the elevation of the initial point would be lower, to be sure, but it would be necessary to raise to the summit only the volume of water required by the summit level and the opposite slope (toward the Seine). Calculations show that the average total of work to be done to raise this water, and, consequently, the power demanded, will not be greater than in the case of obtaining water at Châteauneuf. Since, moreover, the cost of construction would be less, it was decided to establish an electric power plant near the canal, at

about the mid-point of the Loire slope, with a substation at each lock receiving power from this central plant to run the pumps. In spite of the low efficiency of such an installation, it was found to be the most economical plan and to be capable of great adaptability to the variable demands upon it.

PRINCIPAL ELEMENTS OF THE PROJECT.

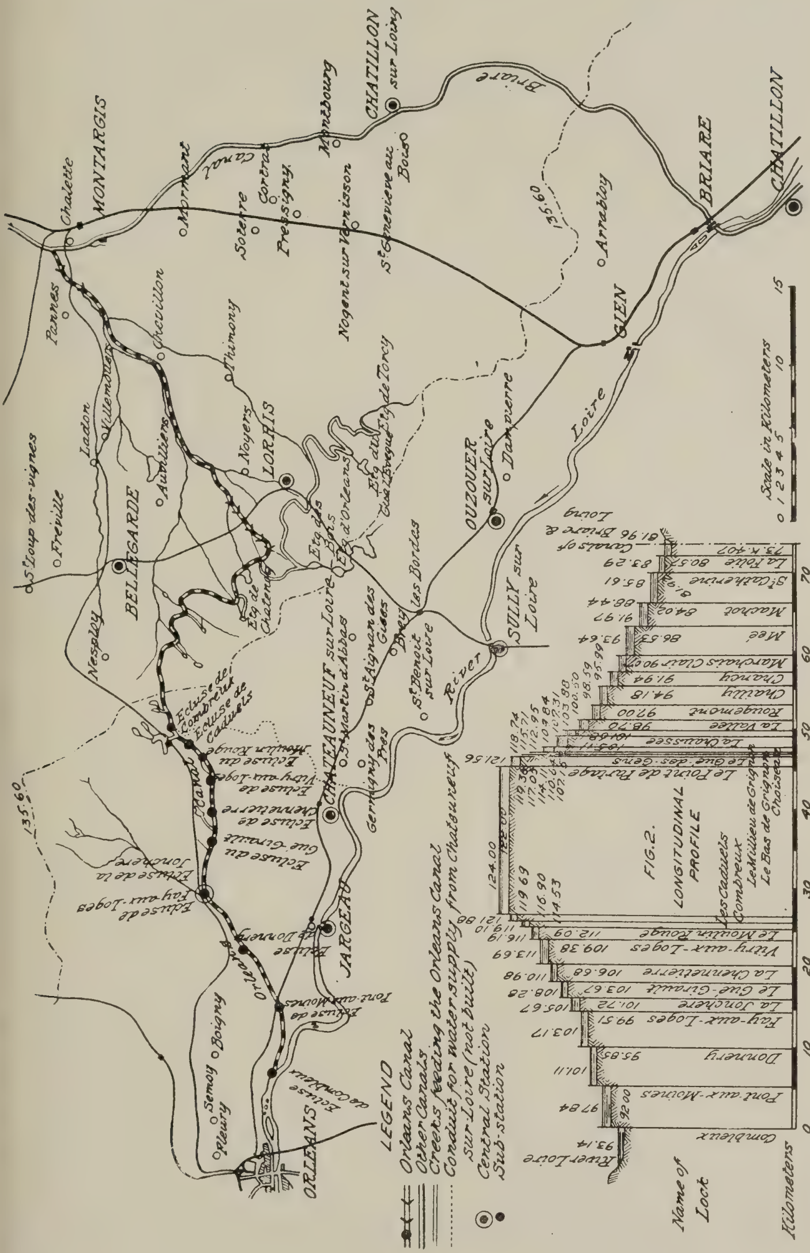
The mechanical and electrical elements have been selected and set down in specifications, of which the following are the chief points:

There is to be a central station near Fay-aux-Loges; one sub-station, with pumps, placed at each of the eleven locks; a twelfth sub-station at the foot of de la Vallée swamp levee, which will be described later, and, lastly, the transmission line.

The power demands upon the various sub-stations are not equal; instead they decrease successively from the lock nearest the Loire, as a result of the losses in the pools and the discharge of tributaries into the pools. On this account the volume lifted may vary from 185 gallons per second at the lowest lock to 114 gallons at the summit level. These figures show maximum requirements; the quantities to be pumped during periods other than those of exceptional consumption will be much less and will vary with the intensity of the traffic and the discharge of the affluents. In fact, each sub-station will consist of duplicate pumping installations, each having half the maximum required capacity, so that not only will it be better adapted for the supply of the normal needs but it also will have, for the same total power requirement, two sets of machinery which, in ordinary cases, may alternate in operation.

The eleven sub-stations at the locks will usually work simultaneously to maintain the pools at their proper levels. However, as soon as any pool receives from tributary streams a water supply sufficient for its needs the sub-station pumping into that pool may be shut down; this will permit the maximum utilization of the natural sources of supply to the canal.

Sub-station 12 is located, as said before, at the foot of la Vallée swamp. This reservoir, the largest of the system supplying water to the canal, is near the summit level. The length of this level is 11.8 miles. It has a clear loss through its sluice gates, during rainy periods, of a considerable portion of the discharge of the drains which enter the pool. The sub-station is intended to turn back this superfluous water into the reservoir, that it may be supplied to the summit level in small quantities as needed during periods of drought. The station will therefore be in operation only at times when the natural water supply is superabundant. For the operation of this sub-station there will be installed in the central power plant a small auxiliary generating unit to avoid possible necessity of operating a large unit at a very small load; this auxiliary unit will at other times be useful for the operation of machinery in the repair shop and for lighting. It should also


LEGEND
Orleans Canal
Other Canals
Creeks feeding the Orleans Canal
Conduit for water supply from Chateauneuf sur Loire (not built)
Central Station
Sub-station
Scale in Kilometers
ORLEANS
JARGEAU
CHATEAUNEUF sur Loire
SULLY sur Loire
OUZOUER sur Loire
GIEN
BRIARE
CHATILLON
BELLEGARDE
LORRIS
MONTARGIS
CHATILLON sur Loing
Loire
Briare Canal
LONGITUDINAL PROFILE
Name of Lock
Kilometers
FIG. 2.

be noted that a reserve will always be held in the swamps to provide against possible accidents to the pumping system.

The necessary capacity of the power plant has been determined from data concerning the amount of water to be pumped and the lift at each lock. It is specified that this capacity shall be divided equally among three units; this measure is justified by the consideration that the installation is intended to supplement the present exceedingly variable water supply. At normal load the plant will supply power for only one of the two pumps at each sub-station; and, moreover, the sub-stations located at the downstream ends of certain pools fed by natural water courses will be idle as long as the combined discharge of these streams is sufficient to supply the demand. The power ordinarily furnished by the central plant will not be as great as one-half, but only about one-third of its capacity.

The annual demand upon the installation will be about as follows: When the natural water supply first falls below the demands of the canal, the reservoirs will be able to supply the deficiency. The power plant will be put into operation when the deficiency becomes great enough to justify the use of a single unit. A second unit will be started when needed, and the third will be held in reserve. It will probably be some time before traffic on the canal becomes heavy enough to require simultaneous operation of the three units. Only then will it be necessary to install a fourth. However, a fourth boiler will be installed at the beginning to permit the frequent necessary cleanings.

CHOICE OF MACHINERY AND DETAILS.

In spite of the broad latitude for details left by the specifications, the ten companies which took part in the bidding presented projects which, from the electrical point of view, are quite similar in their essential details: Three-phase alternators, with a frequency of 50, furnishing the current at 5,000 volts (most of the bidders gave the same figures for frequency and voltage), with transformers and induction motors, operating at lower voltage, to drive the pumps. In reality, such differences as these are between the electrical installations selected by the different bidders are due in large part to the very great variety of their plans for the mechanical part of the project.

The specifications permitted the use of steam engines, turbines, or engines using producer or semi-water gas. From this there resulted thirty-four different projects, of which twenty-three called for the use of boilers and eleven for gas producers. In the first category there were nineteen projects employing steam engines of one type or another and four employing turbines.

The commission in charge of the award decided that if, everything considered, the use of gas did not effect an appreciable saving there was no reason for preferring it to steam. The lowest bids were made by the contractors who had adopted steam, due,

probably, to the location of the plant at a point where the water supply is always sufficient.

Between the turbine and the steam engine proper the choice fell upon the latter, which, in the projects submitted by the bidders, was found less expensive. It seems that turbines are especially adapted for the supply of power in large amounts and that the speed with which they drive the alternators is undesirable.

Finally, in the comparison of the different types of steam engines which were named in the bids, expense of installation was not made the main consideration, as must be the case for a plant whose cost of construction must be paid off in a short time. On the other hand, the desire was to obtain simple, strong engines needing few repairs — and these requirements have apparently been met by horizontal, slow-speed engines.

As to the sub-stations, one of the proposed projects contemplates placing the pumps in wells and driving them by vertical shafts from motors located at the ground level. This system offers an excellent guaranty against accidents which might befall the personnel in case operation of the sub-stations be intrusted to the lock tenders. On the other hand, the more prevalent method of using belt drive permits the driving of the pumps at a speed corresponding to its discharge and lift by simply changing the pulley for one of different diameter while the speed of the induction motor is constant. The number of types of motors and pumps can thus be reduced. It may be seen, too, that if a severe obstruction to the motion of the pump occurs, the belt can slip and the breaking of the metal parts will be avoided. Belt drive, with pump and motor on the same level, was adopted, precautions being taken to assure safety of the operatives from injury by the belt.

DESCRIPTION OF THE PLANT.

* * * Each of the sub-stations on the canal is provided with two Ratean centrifugal pumps and the twelfth, at la Vallée swamp, with a single pump. * * * The mean lift at the locks varies from 6.7 to 12.1 feet. The lift at la Vallée is 3.9 feet.

* * * * * * *

The power requirements called for the installation at the central station of three alternators, rated at 145 kilowatts each. This plant has three horizontal tandem steam engines of 210 effective metric horsepower each, which run at 150 revolutions per minute. The "semi-tubular" boilers are four in number, one for reserve, of which each has a heating surface of 1,080 square feet; they have superheaters and a Green economizer. The three principal generators are three-phase alternators directly connected to the steam engines. * * * The voltage is 8,000 and the frequency 50. * * * The buildings of the central plant provide the room necessary for a fourth and a fifth generating unit.

* * * * * * *

The three-wire transmission line follows the towpath. Its length is 16.8 miles, the central plant being located at the middle point.

* * * * * * *

The lock tenders easily learned to run the sub-stations, the methods of operation being very simple. Each tender is provided with an instruction card giving the steps to be taken in even the most improbable cases of derangement of the apparatus.

* * * * * * *

COST.

* * * The cost of the project may be sub-divided as follows:

Contractor's bid for installation of central plant, transmission line, lighting plant and sub-station apparatus	$120,300
Superintendence, inspection and incidentals	12,100
Buildings (central plant and sub-stations), including purchase of land and damages	64,400
Total cost of construction	$196,800

The estimates for annual cost of maintenance and operation are as follows:

Salaries	$1,740
Maintenance of plant	2,120
Fuel	4,820
Total	$8,680

The idea of the system which has just been described, the preparatory researches, the solution, the detailed project, and its execution are the work of M. Fr. Bonnet, Engineer of the Ponts et Chaussées at Montargis, to whom belongs the credit for the conception and realization of this new method of water supply to a canal. He was ably assisted by M. Dessauny, Conducteur des Ponts et Chaussées at Fay-aux-Loges.

William Price Craighill

William Price Craighill (see frontispiece) was born in Charles Town, Va., July 1, 1833.

He graduated from West Point July 1, 1853, and was then appointed Brevet Second Lieutentant in the Corps of Engineers.

His first service was in Charleston, S. C., in Savannah, and at the Dry Tortugas, Fla., during the period after graduation until 1856, when he was ordered to Washington as assistant to General Totten, then Chief of Engineers, where he remained until 1859.

He was promoted to First Lieutentant July 1, 1859.

From 1859 to June 23, 1863, he was on duty at West Point as Assistant Professor of Engineering, part of the time as Treasurer and in command of the Engineer Detachment. During this period, in 1862 he served in the field as Chief Engineer of Gen. G. W. Morgan's Division of the Army of the Ohio, engaged in the defense of Cumberland Gap. His services on this occasion brought forth the following commendation from General Morgan: "Nor can I close this report without calling the attention of the Commanding General to the important services rendered me by Lieut. W. P. Craighill of the Engineer Corps. He is an officer of distinguished merit and is thoroughly informed on all subjects connected with the art of war. He would make an able chief of staff or fill with high credit any other position to which he may be assigned, and deserves a much higher grade than he now holds."

In June, 1863, he was detailed to fortify Pittsburg, Pa., the citizens of which were in fear of a raid by the Confederates. In about ten days he built a complete line of defenses around the city by voluntary labor and contribution of citizens, without help either of men or money from the General Government. This was a remarkable achievement and shows what may be done under stress.

He was promoted to Captain on March 3, 1863.

After completion of the duty at Pittsburg, he was for a few weeks in the office of the Chief of Engineers, and then from September 16, 1863, to June 13, 1864, in Baltimore as assistant to Colonel Brewerton, serving a part of the time as Chief Engineer of the Eighth Army Corps and of the Middle Department.

It was during this time that he located the channel in Baltimore Harbor that, at Colonel Brewerton's suggestion, was given his name, thereby establishing what has become the most enduring monument to his skill as a civil engineer. He then conceived the idea of locating this channel on the line of the resultant of the forces of the currents acting upon it. The principle has been amply justified by the manner in which this channel has maintained itself.

In August, 1864, he was sent to California as a member of a special Board of Engineers for the defenses of San Francisco. This was on account of the danger of war with France over the Mexican situation. He remained in California until the spring of 1865, when he was detailed on a similar board for the defenses of New York, remaining on this duty until November 10, 1865. He was promoted Major, November 23, 1865, having received the brevet rank of Lieutenant-Colonel on March 13, 1865, "particularly for services in the defense of Cumberland Gap and the ulterior operations of General Morgan's forces."

He declined the further brevet of Colonel.

During the years from 1865 to March 31, 1870, he was again on duty in the office of the Chief of Engineers and in charge of the Baltimore office. From April 1, 1870, until his appointment to Chief of Engineers, May 10, 1895, he remained in Baltimore. During this period of twenty-five years his duties were greatly varied, including at times the works now belonging to the Wilmington, Del., Baltimore, Washington, Norfolk, and Wilmington, N. C., districts, besides the Great Kanawha River. In connection with this last work he made a trip to Europe in 1877-1878 with Colonel Merrill to study movable dams and other works of internal improvement in France and Great Britain. He commenced and nearly finished the Kanawha River improvement, and saw it finished and in successful operation under General Hains—his successor in the Baltimore office. He was promoted Colonel on January 10, 1887. About this time, on July 1, 1884, the present system of division engineers was inaugurated by giving him supervision of junior officers in Delaware, Maryland, Virginia, and North Carolina, with the title of Supervising Engineer. This arrangement continued until December 3, 1888, when the present organization was adopted. During his service in Baltimore he was on many of the boards for the most important projects for

the coast defenses and river and harbor improvements in all parts of the country.

He had a tremendous capacity for work and never failed to give his close attention to the details of whatever was committed to his charge, whether small or great. An instance of this was the establishment of the camp for the centennial of the surrender of the British at Yorktown, Va. This was the first great camp in which attention was given to perfecting the arrangements for sanitation, water supply, etc., in the manner of our present concentration and maneuver camps. His work on this occasion called for a special letter of commendation from General Hancock, who was in command of the troops.

He also built the beautiful monument which now commemorates the victory at Yorktown. On May 10, 1895, he was made Chief of Engineers, with the rank of Brigadier-General, and retired two years later at his own request, on February 1, 1897.

During his term as Chief of Engineers he started the work on the present system of coast defenses, arousing by his personal efforts the interest of the Secretary of War and Congress in the matter, so as to obtain funds for beginning the work in an adequate manner, as well as by establishing some of the most important features of the works as since carried out.

After his retirement he refused civil employment, except for a time as member of the Consulting Engineers to the Dock Department of New York.

He took a lively interest in the American Society of Civil Engineers and was one of the first Army officers to join that Society. He was president of the Society in the year 1894-1895. He regarded this as one of the highest honors of his life, and directed that the fact be inscribed upon his tombstone. After an illness of nine months, he died January 18, 1909, in Charles Town, West Virginia, where he was born and where he is buried.

Some Recent Tendencies in Field Engineering*

BY

Capt. E. E. B. HOLT WILSON
D. S. O., Royal Engineers

TYPICAL TACTICS FOR RESISTANCE.

On active service there arrives a moment when theories and text-books fade into the background, and the young field engineer finds himself face to face with the execution, and not the mensuration, of field defence works. The value of good organization and training, sound strategy, capable commanders, efficient troops, and the lessons of military history, have already found definite expression in the instructions given to him by his immediate superior, and the field engineer is in point of hard fact now required to deliver the finished article, ready for use, and situated on ground unlike any of the favorable examples he has previously encountered in print.

It is chiefly in matters of detail, especially in the correct variation of the normal to suit the tactical occasion, that the officer of field engineers is able to show that he intends his side to beat the enemy, not merely to baffle him.

The particular tendencies of field engineering which it is proposed to examine now are those of the technical design of certain of these details of fieldworks, more especially as disclosed in the recent publications of foreign countries.

All armies have normal types for fieldworks. They are based upon the tactics considered most likely to lead to victory over the most probable opponents; and in Continental manuals of engineering the types are planned almost entirely for use in the particular variation of the offensive-defensive which it is expected will be employed against highly trained civilized troops—the situation, for instance, of our Territorial Force employed for home defence.

We have become so accustomed to acknowledge that the correct tactical application of fieldworks is everything and the details nothing, that we are in danger of leaving it at that, and of trusting entirely to such flashes of Napoleonic genius in matters of detail on the part of all ranks as we already demand from the higher commanders upon the momentous question of when to launch their general reserve beyond recall. We must remember, however, that tactics themselves are in part only the methods employed to get the

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most effective shooting out of a given force, and that the shooting itself is the thing that matters. As the principal function of all field engineering—properly applied—is also to get the most effective shooting from the smallest possible given team, we are on clear ground at once and can proceed to details.

The days have now passed when entrenchments were issued by the yard, run conformable to sealed pattern—in peace time only too frequently as tortuous exercises in mensuration at paper examinations, and in war as "some fad of the engineers," to be carried out under protest by the unwilling troops. Redoubts, even, are no longer planned at the office table and served out in stock sizes, like hats, to be crammed somehow on to the crown of hills, many of which have nasty habits, in the Quartermaster's language, of turning out to be "specials." But, though we beware of stock type-designs and stock systems, it does not mean that we go hatless because the sealed pattern article does not fit our head.

The spirit of the offensive rightly permeates all manuals of field engineering to-day, and good normal details should accordingly possess inherent adaptability to the purpose of sound offensive tactics. It is therefore difficult to choose a normal tactical peg on which to hang samples of fieldworks; for current tactical conceptions of their use as an aid to good shooting have such a wide range. We have a good ideal in the Japanese idea of placing rows of dummy parapets on a false position, holding them with just enough men to make a sufficient show of resistance to cause the enemy to deploy for attack, and then hurling on the flank of the attack the whole nation in arms—hitherto concealed round the corner! But perhaps this might be a little too advanced for our present purpose.

STRONGHOLDS OF DISPERSED ELEMENTS.

Leaving the main striking forces out of the picture, it will perhaps be safer to assume that the defensive share of the fighting is to be done by a relatively small number of troops *placed in charge of*, not "posted for the defence of," a zone of resistance, which, starting from the smallest beginnings, develops its strength from hour to hour to meet the particular stresses to which it is being subjected. The backbone of such a defence may consist in some of the troops judiciously posted in a chain of mobile garrisons, of strength proportionate to the estimated difficulty of their tasks, and within co-operative distance of each other. Their business is not to repulse attacks but to encourage them, and to be a terror to the neighborhood; to be a hornet's nest when stirred with a stick, and yet able to roll themselves up into the prickliest ball of a hedgehog that ever baffled a pack of terriers. They should ensure that the hostile scouts do not report that the enemy are entrenched at such and such points, but that "the country is infested with the enemy."

The underlying principle is that every man and gun should be

continuously employed so as to immobilize the largest possible number of the enemy, keep them out of mischief elsewhere, get their teeth into them, and at the same time make it impossible for them either to withdraw, or even to maintain their ground, without a steady drain upon their reserves.

The depth of such a zone of resistance should be sufficient to provide inducements for the enemy to develop an irregular and broken frontage, deficient in lateral cohesion, tending to present local flanks vulnerable to local attack by local mobile reserves, and difficult of support by artillery. In brief, to catch flies in detail by enticing them to walk across a wide strip of sticky paper instead of rebuffing them with a wire gauze screen, without catching them.

In order to produce the necessary distortion of the attack; to afford foothold for these garrisons in times of extreme stress; and to act as pivots of maneuver within the zone of resistance, the consensus of foreign opinion points to the selection and fortification—probably in the form of miniature field fortresses with definite garrisons finding their own local reserves for offensive action—of a certain number of small areas which are likely to become or to contain points of possible future tactical value to the enemy. We can not, as a rule, label them "tactical points" by virtue of any topographical consideration alone, for "tactical point" is not a geographical expression, but one compounded of time *and* place—which may be defined crudely as "the piece of ground the enemy would like *next*." Fighting a zone of resistance on chance ground in the open field bears to a strategically chosen and deliberately fortified fortress area much the same relationship that a point-to-point race across unflagged country bears to flat racing between the rails, and it requires a constant succession of bold and rapid decisions and ceaseless activity with the spade. Granted that, for our present purposes the zone of resistance will include the provision of some self-contained entrenched groups, strongholds, or redoubts, we will examine their probable disposition and design.

The first and principal lesson taught us as field engineers by the Russians in Manchuria is the failure in defence of a linear disposition of fire trenches closely supported by rear lines of trenches or redoubts which are not fully engaged until a portion of the original front line has fallen. The center of gravity of the defence has moved forward from its old position on the lines of strong closed supporting works in rear right up to the very foremost trenches, and has taken the strong works with it. "Foremost" is not here used in a linear sense, but in the sense of having the pick of the early firing points, a step-by-step abandonment of which is not contemplated.

Such a stronghold or group then must fulfil two distinct purposes: to be the headquarters of an offensive hornet's nest by day, and the prickly hedgehog during fog, darkness, reduced garrison, or the heavy rush; and all the time it must take a steady toll of the

enemy without letting him attain his object, namely, to open secure communications through the zone of resistance. Any attempt to rely upon one general disposition of troops to deal with both sets of conditions, especially that suited for hedgehog tactics alone, has usually led to failure. For dealing with the enemy by day we must have, in addition to artillery, numerous alternative, invisible shooting pits and small but deep trenches, to hold from four to fifty rifles each, well disposed to get the pick of the shooting, with plenty of small bolt-holes and shelters, and all in covered communication with each other and with supports and local reserves.

For passive resistance during the close seasons of darkness we must arrange that the enemy, before he can overrun the group, *must* pass over our section of near foreground which has been marked out in a series of well-fenced death alleys, every foot of which can be brought under a deadly enfilade fire from firing points which are manned for this purpose alone and, if possible, entirely screened from the direct fire of the enemy. Such firing points are often designated as ''stabbing'' trenches or ''stabbing'' batteries.

The measures outlined above form, however, but the second of the important activities of the garrisons of a zone of resistance. They have been selected as being more closely associated with the heavy work of the field engineer than are the primary services of protection by reconnaissance, which provide the forewarning without which forearming is futile.

An advanced screen or false front forms a connecting link between the measures of protection by reconnaissance and of protection by active resistance. Its preparation may often call for greater skill in treatment by the engineer than the main zone of resistance itself. Great importance is attached in most Continental armies to the use of an intrenched false front, not only as a means of obtaining indications of the enemy's intentions and dispositions, but as a means of gauging his powers and methods of attack, and of inspecting his armory generally, in sufficient time to arrange a suitable treatment for him in the main zone of resistance. If his artillery is to be brought under review, guns must form part of the advanced screen also.

But although the engineer, thinking perhaps of the unreadiness of his freshly planted ''concealments'' and unfinished communications in the main zone, may be tempted to put such good work into a false front that, with modern weapons, this front may unexpectedly hold its own with ease as a barrier screen for a considerable period, such success may not always further the intentions of the commander to get his teeth into the enemy and worry him in the main arena. Such an unintentionally severe check to his advance may end in so sudden a change of his plans that not only will the Intelligence Department of the defenders be hard put to it to overtake his new movement, but their engineers may

Fig. 1. Left half. Example of field stronghold or group (field defenses and mobile artillery). Garrison, about one battalion.

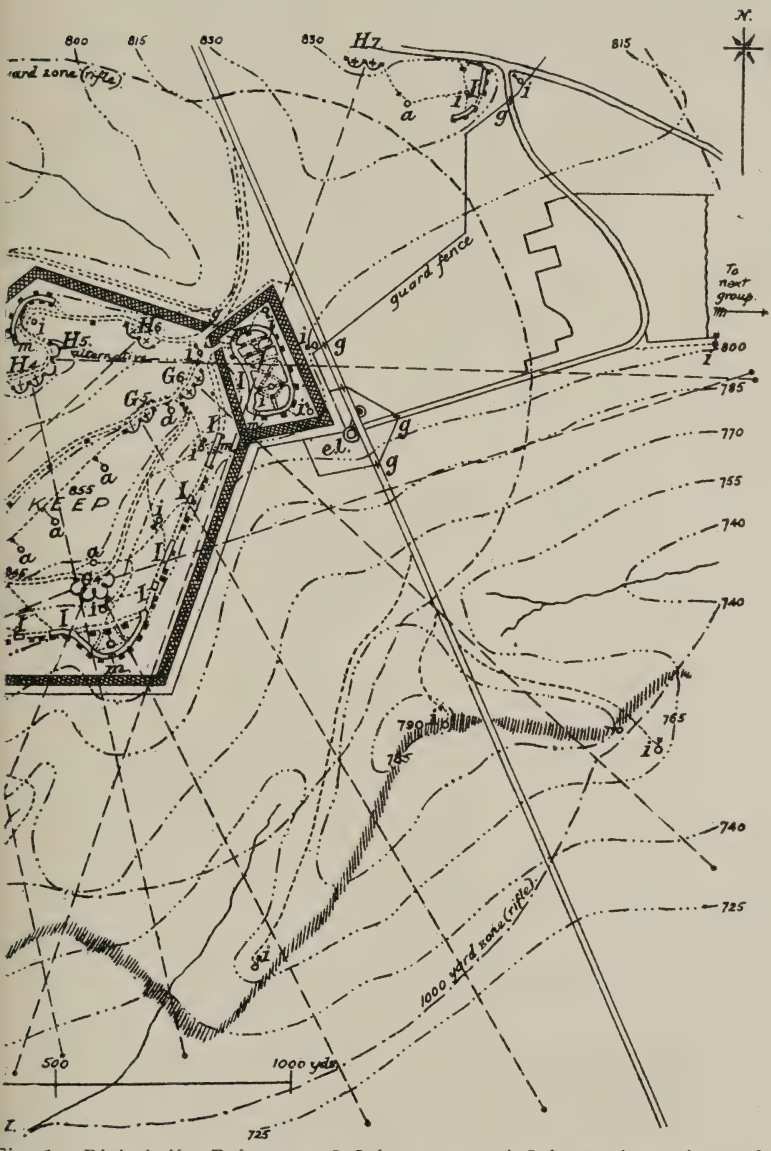


Fig. 1. Right half. References: *I*, Infantry post. *i*, Infantry observation or fire control post. *G*, Guns. *H*, Howitzers. *a*, Artillery observation post. *e. l.*, Field searchlight and shelter. *m*, Machine gun. *g*, Gate in obstacle. ...., Splinter-proof shelter. ::::, Main communications. / / / /, Dead ground (rifle fire). Note: Batteries H_2, H_5, and G_4 are in alternative positions, which are manned only for the close assistance of the adjacent groups.

scarcely have time to dig and roof the next series of fighting pits, still less to complete the gardening of the peaceful landscape with its steel wire undergrowth.

If such a screen is formed in front of an entrenched zone of limited frontage it appears to be advisable to keep its main positions well out to either flank. This may cause a greater and more premature dispersion of the enemy in deployment than the original frontage of the main zone would have done, and will increase his circuit for the turning movement which is awaited in all positions of limited frontage; while the withdrawal of the screens will leave them still playing the part of echelon supports to resist the envelopment of the main zone round which they pivot, and at the same time decoying the enemy into well ''treacled'' ground.

REDOUBTS.

It should now be easy to realize why the word ''redoubt'' has almost acquired a guilty sound of defeat in the soldier's ear. In its hitherto accepted form and meaning it has been associated almost exclusively with the form of resistance which may be described as the ''rolled-up hedgehog and hope for the best'' policy—the last resort of the half-beaten soldier. The principal defects of the small fort or redoubt which stood alone, excluding the tactical aspects of its employment, were one or more of the following: relatively short and formal frontage of continuous fire parapet, still shorter flanks, often traced from arbitrary or even geometrical considerations to deliver a ''defensive'' flanking fire, regardless alike of probabilities and of the demands of adjacent works for particular lines of supporting cross-fire. It as a rule occupied one small topographical feature, and that imperfectly, and often added to this vice that crowning crime of visibility from afar which has been said to form one of the chief safeguards of some of our defended ports against oversea invasion—because the cry of ''Look at their forts'' might send a battalion of the enemy with such a rush over to one side of the vessel carrying it that it would capsize in deep water. The small formal redoubt can perhaps be seen in its most pernicious form in some of our earlier designs for that ill-mated partnership—a battery for heavy guns and a flank and gorge parapet of formal trace for so-called ''musketry defence,'' the whole surrounded by an unflanked obstacle of oval trace, just as it came out of the standard-plan cupboard. Surely the zenith of tactical confusion in the application of fortification to war!

The Russians realized much of the foregoing in Manchuria when it was too late to change their tactics in passive resistance; for to-day a change of method in the application of field fortification is no longer merely a change of trace of profile, but affects the fundamental training of the troops who are to employ this weapon. The Russians tried, in their field entrenchments, to put their formal redoubts right by adding wing or whisker trenches to increase the frontal fire, but were too late in realizing that the closed works

themselves should have been right up in the front fighting line, and of tactical, not formal, trace.

It was not until the close fighting had set in at Port Arthur itself that they endeavored in costly haste to relegate their monumental permanent forts to their only possible rôle as "keeps" to a series of modern entrenched groups, and ploughed the rocky foreground with numberless fire trenches in the attempt to meet the advances of the enemy on even, if not superior, terms. But the ardor of the enemy's attack and the rocky soil were against them, and their groups never developed sufficient strength to perform their task effectively, either in mutual fire support, intercommunication with the main arteries, or in dispersion and multiplication of targets to dilute the enemy's heavy fire.

What can be done in the way of resistance, even by an indif-

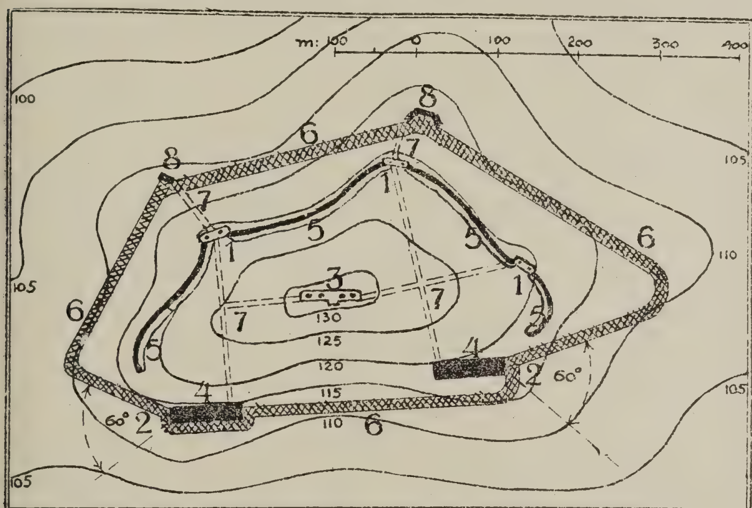


Fig. 2. Example of permament stronghold or group (Austrian). 1. Battery for 2 Q. F. guns or Q. F. howitzers. 2. Stabbing battery of 4 to 6 inch guns in armored casemates. 3. Battery of long range guns. 4. Masonry shell-proof shelters: musketry gorge. 5. Fire position for infantry and machine guns. 6. Wire entanglement in open ditch, graded for enfilade fire. 7. Shell-proof underground passages. 8. Sunken flanking galleries.

ferently entrenched but well-fought group-stronghold, is strikingly exemplified by the case of the twin redoubt group on the 210-203 Meter Ridge. Here a garrison of 1,500 Russians, renewed until they had suffered 3,000 casualties, inflicted a loss of upwards of 14,000 men upon their still braver attackers before the group fell.

There are three typical positions for the employment of invisible redoubts in a group stronghold in the first line, of the type under consideration. Not being limited, as a rule, to one topographical bump on the ground, the extent of frontage taken up by such a group can often be lengthened and can be given the pick of the

firing points for frontal fire until favorable ground is found to develop a really effective flanking fire across the adjacent groups. All flanks are a source of anxiety and here is one typical position for the use of redoubts—redoubts with their main lines of fire devoted to the control of the intervals, and their own flanks and gorge traced for home use in the passive defence of their group in times of stress. The second typical position for a redoubt within a group is such as to stiffen a salient, if any, in the main front of the group—but here there is a divergence of opinion—the Germans put a redoubt in the forefront, but the French prefer the third typical position, namely, behind the skyline as seen by the hostile artillery.

The merits of the second and third positions are questions which we have not time to discuss at present and are, after all, dependent upon local circumstances. Which redoubt should be selected for early strong treatment to form the key or keep of the group depends more upon the enemy's tactical facilities in attack than upon local topography, though the importance of developing the full *offensive* powers of all works from the first has already been alluded to.

The general treatment of the intervals between such groups appears to be the provision of a chain of selected firing points well echeloned back in a concave festoon in plan, entrenched and concealed, but only lightly picketed with troops; ready to receive fresh troops if called upon to check a serious attempt to punch a hole through an interval; as a screen in front of lateral communications and of batteries posted in support of the groups, and for controlling ground dead to the fire of the groups.

We will now examine a few types of fieldworks which may help to illustrate some of the recent tendencies in further detail.

A Sketch of a Field Stronghold or Group, planned for Field Defences and Mobile Artillery (Fig. 1).—There are twelve alternative concealed pits for direct-fire guns, of which four are for emergency use in the defence of the salient. Ten alternative emplacements for field howitzers, of which two (marked H6) are set back to develop fire upon dead ground in the S. W., on the 500-yard rifle zone, and four (H2 and H5) for the special assistance of adjacent strongholds. Redoubts at the flanks: A second tier of direct fire as a salient keep. A wire obstacle enfiladed by machine guns and rifle fire. Echelon support is derived from other mobile guns and howitzers, and an additional obstacle for artillery enfilade is provided on the salient spur to the west. Liberal communications are provided throughout the group, and numerous small shelters in the firing line, redoubts, and reverse slope communications, each to hold about five to ten men. Two field emplacements are provided outside for mobile electric lights, with the usual artillery and infantry fire control and observing stations. A guard fence is shown to assist in the control of the intervals at night, and to defer a near approach to the main obstacle. The group occupies

about 150 acres of ground, an area about equal to that of the "Feste Kaiserin," one of the satellite strongholds of the Fortress of Metz.

**A Permanent Stronghold or Group (Austrian)*. (Fig. 2.)—The backbone consists of three pairs of armored Q. F. guns for direct fire, and four long-range direct-fire armored guns on the highest point. The fire position for infantry is in the form of a curtain trench between the Q. F. guns, which are at the salients. At the gorge are two masonry shell-proof shelters which provide reverse musketry fire, and gorge stabbing batteries for flanking the intervals. The obstacle, which is in an open, carefully graded ditch, is enfiladed from sunken flanking galleries, and by the gorge shelters.

A Simple Group, or Extended Redoubt, Field Entrenchment

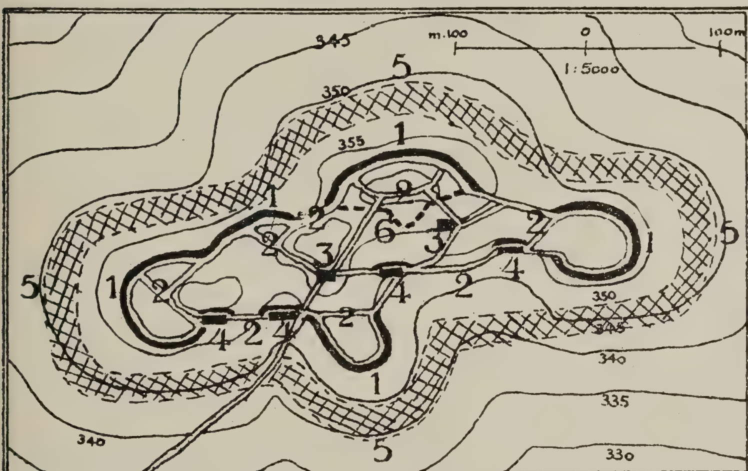


Fig. 3. Example of simple group (Austrian). 1. Splinter-proof fire trench. 2. Communication trenches. 3. Masonry shell-proof shelters. 4. Shelters proof against field howitzers 5. Wire entanglement. 6. Keep.

(Austrian). (Fig. 3.)—This is an example of a redoubt taken to pieces and distributed to the best advantage for fire effect. There is continuous communication but not continuous fire trench. Six shell-proof shelters in masonry would classify it as a "provisional" work. A "keep" is formed at the central forward salient, marked 6. The wire obstacle is more or less well flanked.

German Field Redoubt for One War Company (220 men). (Fig. 4.)—Plain oval trace, 50 meters front to rear. Unflanked obstacle, and outlying sentry pits. Forty-four shelters for five men each provide shelter for all the garrison. The latrine and dressing station are on the forward slope, at 5 and 3, and the headquarter telephone shelter on the reverse slope at 4.

An Example of Fire Trenches Refusing a Flank in Echelon.

*From *Permanent Fortification for the Imperial Military Training Establishments, etc.''* by Mr. von Brunner, Major, Austrian Engineer Staff, 1909.

(Fig. 5.)—Echelon support is considered better than "turning back" the flanks whenever possible, as it increases the circuit of the necessary outflanking movement. The unit trenches are similar in principle to the example in the "Manual of Field Engineering," 1911, and portions of the support trenches, under cover from the front, are prepared to act as stabbing trenches for flank fire only. One machine gun is shown flanking the forward slopes of an outlying spur, and one in a stabbing emplacement, to sweep the crest and reverse of the spur. A temporary end is formed with a closed work and stabbing battery, and is supported conventionally by mobile reserves of three arms, and covered by heavy artillery still farther back, and not shown on the sketch.

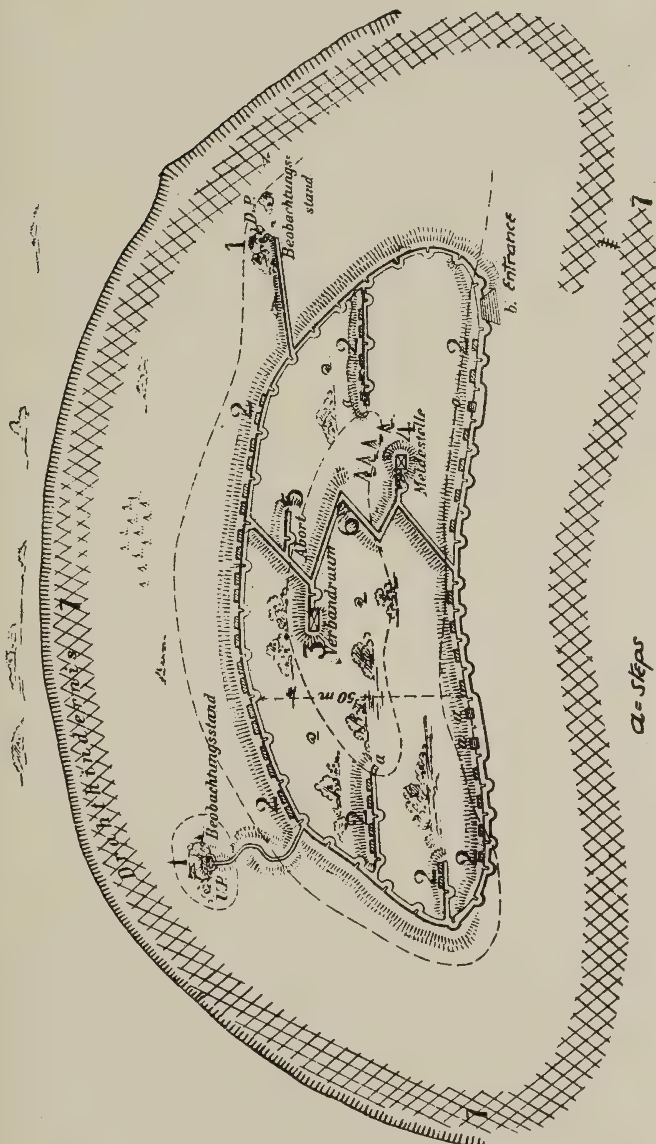
PROTECTIVE PROFILES.

Profiles of fire trenches are not illustrated in text-books as a prescription for effective shooting, but as a normal and economical provision for protection, concealment, and communication under certain average conditions of time, labor, soil, site, and field of fire which are seldom all found in conjunction; and it is extremely unsound to lay down a given profile to be adhered to, even throughout the length of one small trench. To lay down or report in a paper scheme that a whole position would be entrenched, and shelters made to a given profile (not unknown in the past), would show a disregard for death and local peculiarities which might be magnificent, but certainly not war. A fundamental rule in defence must always be that *each* rifleman is entrenched to the best advantage to give a good account for every cartridge he fires: and whilst inside his lair must not get killed except by accident. Beyond that it is of little interest to know how many cubic feet of earth he disturbs for the purpose: though it is often of interest to know that it will average so many hours *per rifle* to arrive at the required result. Even this can only be a wild guess before at least a portion of the given sites have been actually dealt with under the war conditions prevailing at the moment. The differences of time from the normal, caused by war factors, are more often in days and weeks than in smaller units of time. A sound rule in such estimates of time is to add twenty-four hours (to the calculated answer) for the "King's enemies," earthquakes, rain, etc. Remember also that "easy soil" means more excavation, more revetment, more return filling, and generally *more time* to arrive at a given profile or degree of protection than in ground firm but not rocky.

It may not be without interest to glance at some typical normal profiles of fire trenches from foreign armies.

Normal Profiles for Fire Trenches (Fig. 6) *as used by France, Russia, Germany, Japan and Great Britain.*—Fire standing is normal for all. The arbitrary effect of local units of measurement is evident in the figures selected for describing a suitable average height over which to fire standing on level ground. Rus-

FIG. 4.—Example of a Low Command Redoubt German



1. Lookout. 2. Shelters (44 at 5 men each). 3. Dressing station. 4. Telephone and signals. 5. Latrine. 6. Passage. 7. Wire obstacle.

sians fire over 4 feet 8 inches (2 arshin); France and Germany, 4 feet 7⅛ inches (1.40 meters); Great Britain, 4 feet 6 inches; Japan, 4 feet 3 1-5 inches (1.30 meters).

The height (1.30 meters) for normal "fire standing" adopted by the Japanese is that considered suitable for a man 1.54 meters in height (5 feet 0.63 inch). Taking this proportion (five-sixths of the man's height) we find that 4 feet 6 inches corresponds to a height of 5 feet 5 inches nearly, and that a Guardsman of 6 feet should fire 5 feet on level ground.

Four types of profile are illustrated, and the average cross sectional area in square feet is given above each type. The average time to *excavate only* 2 paces of the average profile of each type, at British normal rates of work, is as follows: Type 1, without parapet, three and one-half hours; type 2, with parapet, one and one-half hours; type 3, deepened, five hours; type 4, kneeling, fifty minutes. The designers, however, claim a higher speed in their text-books.

Points of interest are that Russia and Japan have adopted a normal parapet command of 1 foot 9 inches and 1 foot 8 inches, as the result of a stern experience of average values; and have cut the width of their revetted firing step down to 1 foot 2 inches and 1 foot 4 inches to get the utmost value from the cover of the interior slope. Many other points of comparison could be drawn and discussed. On the whole, our types appear to be on the wide side of the average (and would be wider still in soft ground) and unduly exposed to plunging fire on account of the great width of elbow rest adopted and the low height (4 feet 6 inches) fired over compared to the other European armies of equal and, in some cases, lower average stature.

The Russians and French leave the question of elbow rest for local adjustment by the rifleman. The former, however, provide a cartridge berm, of the width of the hand, at ground level, and invariably shoot over fire cover with the *left hand on the butt* of the rifle: thereby deriving the last ounce of cover and steadiness from the parapet.

In type 3, with a passage trench in rear of the firer, the British normal type shows the greatest total vertical depth of cover (6 feet 6 inches), which is attained by the drop of 2 feet from the firing step; as compared with France and Japan, 1 foot 4 inches; Germany, 1 foot 8 inches; and Russia, 1 foot 9 inches. The Russian type also shows a parados as a normal provision, partly in order to reduce the area of parapet to be concealed in front, and partly for reasons indicated presently.

One of the duties of a normal profile is to suggest ideals, and the representation of a vertical interior slope by France and Great Britain serves as a reminder of the importance of reducing to a minimum the horizontal distance between the foot of the interior slope and the firing crest above it.

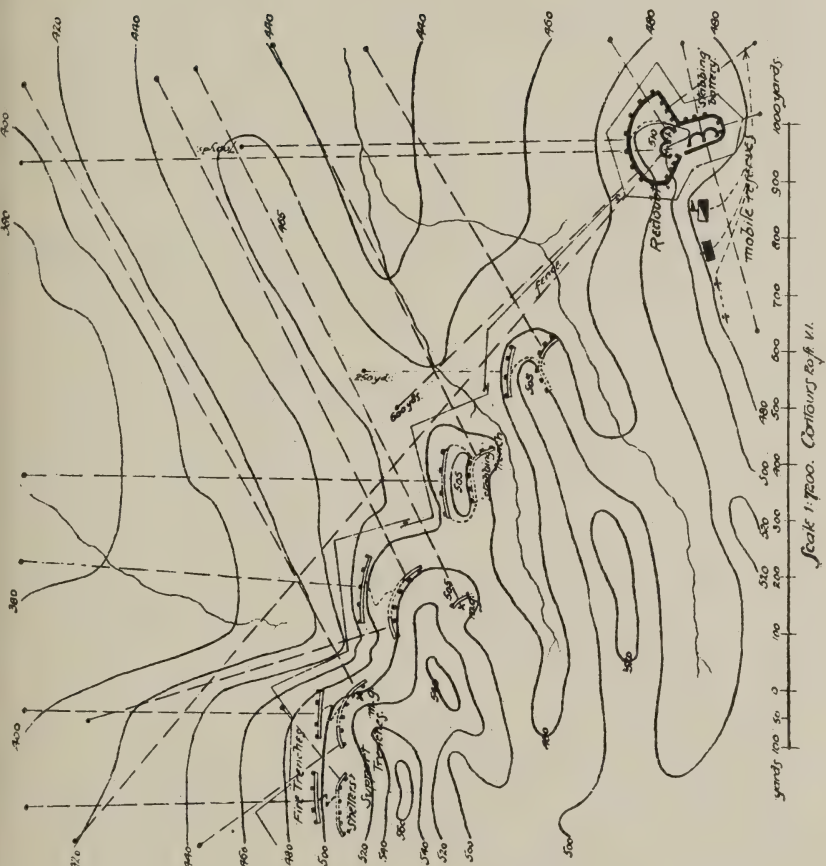


FIG. 5.—Example of Fire and Support Trenches in groups of about 50 rifles; and the method of gradually refusing a flank in echelon.

ENTRENCHING UNDER FIRE AND IN THE ATTACK.

All manuals deal cautiously with this subject, for fear of training their infantry to entrench every attack to a standstill.

The French say "a trench dug during the attack must never become the grave of the offensive spirit."

It is rather a pity that the subject is somewhat shirked, as we all know very well that it will probably only be by means of the spade that any modern troops will be able to make good their advance upon troops already entrenched, or even behind better natural cover. The subject is, however, almost entirely a tactical one and in technical details falls under an introduction to the subject of siege works.

The Russians have shown a distinct tendency recently in favor of the use of sandbags in the attack as a preferable alternative to grubbing with the entrenching implement. A suitable bag is said to measure about 20 by 14 inches, with 40 pounds of gravel. Colored gray-green, it is said that a man is invisible behind it at 1,100 yards in the open, and that even at 400 yards the target is extremely difficult to pick up. It is also claimed that as it can provide bullet-proof cover the men much prefer the labor of dragging it along 30 or 40 yards at a time to the unsatisfactory process of frequently grubbing in the ground only to provide a very visible and non-bullet-proof screen—for bullet-proof cover is, as a rule, out of the question in this class of work. There is also considered to be less danger of the attack entrenching itself to a standstill.

A German comment on this principle claims that the occasions for favorable use would be exceptional, and that it would diminish the men's activity and cripple the offensive spirit—remarks which would be equaly applicable to the provision of any entrenching implement at all for use in the attack, and which appear to point more to an optimistic estimate of the bravery of the soldier than to the recent experience of the stern realities of war which underlie the Russian experiments.

PROTECTION IN THE FIRING LINE FROM ENFILADE AND REVERSE FIRE.

The full importance of effective traversing from oblique, enfilade and reverse fire is not, as a rule, properly appreciated, except when under fire. One has only to study the development of details as a war progreses to notice the general tendency toward placing every rifleman under a lid and in a safe tub of his own, for shooting from; and in a still safer bolt-hole nearby for protection when he is being "over-shot at." Whether it is that in peace we do not realize the full range of modern weapons—for the German rifle kills at 4,400 yards (2½ miles), or that we are over-sensitive in admitting the fact that little mistakes such as "shorts" from supporting artillery, or a terrific outburst of rifle fire from overexcited supports are constant occurrences in war, and must be insured against. The fact remains that when trigger-snatching be-

gins in earnest streams of metal will, and do, arrive from absolutely untraceable sources.

From starting without any traverses to speak of, the Russians finished up in most places with a recess for every rifleman, and heavier "passage traverses" from 8 to 15 yards apart, according to the degree of obvious exposure. Back cover is provided always. This may be raised high enough to provide a background if necessary, but no traverse is ever allowed to show above the general level of the parapet, and, especially on a forward slope, every traverse is made to fall to the rear at a steeper slope than the line of sight from the enemy's gun positions.

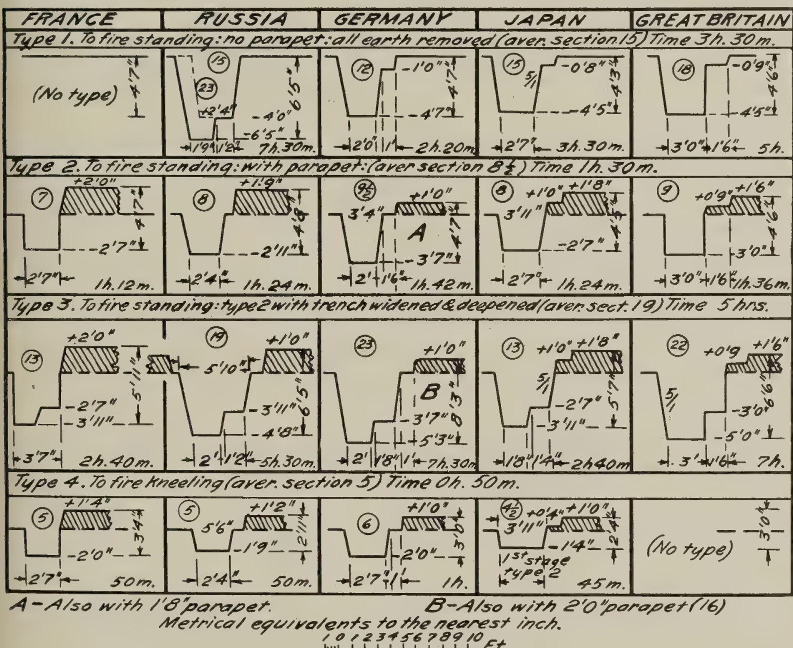


Fig. 6. Normal profiles for fire trenches, approximate sectional area (sq. ft.) shown in circles. Time required to excavate 5'0 lin. ft. at normal British rate of work, shown in lower right hand corner of each figure.

We find also in the final Japanese lines north of Tiehling the men were recessed in twos and threes, and similar traverses were used.

A moral value of many small traverses is that they often save men the knowledge of what has happened in the next compartment. Density of fire line has ceased to affect the question seriously, since the Russians tell us that *trained* riflemen well entrenched from 5 to 8 feet apart can deal with the worst situations by day or night, and it can usually be arranged to develop an emergency shoulder-to-

shoulder line, if desired, to fire over any parados or head-cover at night.

PROTECTION FROM PLUNGING FIRE.

About the value of good head and overhead cover as an ideal there are no two opinions, and Lieutenant-Colonel von Swartz, of the Russian Engineers, writing while the lessons of Manchuria were still deeply bitten into his Sapper conscience, says:

"The designs of the days of infantry parapets without head-cover for the rifleman must now be put away among the curiosities of ancient history." The Japanese, Russians, and Germans all agree that overhead shelter from splinters and bullets *must* be provided somehow *actually in the firing line.*

It may be taken as an axiom that a rifleman detected firing from behind *visible* cover is at an actual disadvantage as compared with an undetected opponent who is making intelligent use of natural ground cover; and the former may as well, if he has the time and material, roof himself in straight away.

But before any form of head-cover is undertaken as a *precaution in advance* the following question has to be answered in the affirmative: Shall we have the time and the means to render our *head-cover* indistinguishable from our background at the distance at which it may have to face the enemy's artillery? If the answer is no, or even doubtful, it would be wiser to dismiss the idea as tending to weakness rather than strength, and to put the labor into small and numerous shelters *inside the fire trenches,* to deepen cover generally, and to make sure of good communications and concealment.

Once we have definitely been found out and ranged by both gun and rifle, and concealment has failed us, in the absence of alternative concealed cover, our next move is to multiply targets incessantly, and to strengthen our cover regardless of concealment, including if possible the addition of *overhead cover,* and so to dilute the enemy's fire by offering such a bewildering choice of supicious objects that the smile is once more on our side. Or we may follow this policy from the first, as most other armies do, if we can not trust to concealment. On the other hand, there appears to be no excuse for omitting well concealed overhead cover in strategic defences deliberately prepared from twenty-four hours and upwards in advance.

We can now examine some examples of cover in fire trenches.

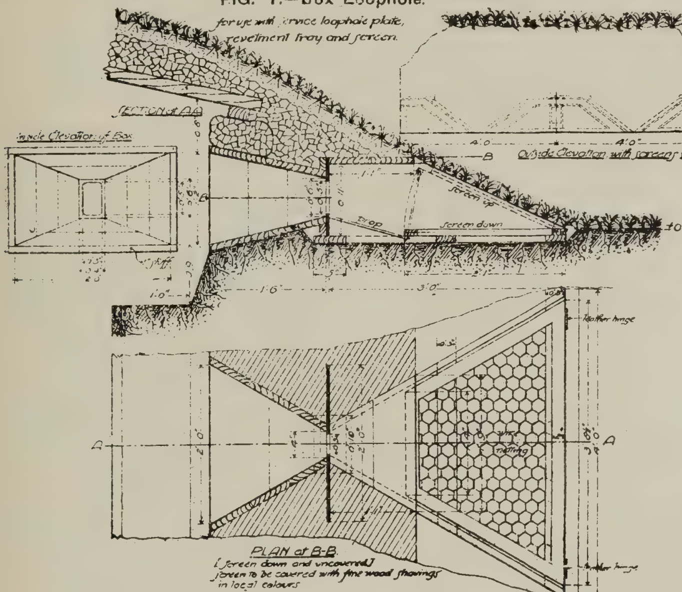
A Type of Box Loophole Suitable for Use With or Without a British Service Loophole Plate in Deliberately Concealed Works. To give 60° Arc of Traverse (Fig. 7).—The result of experiments to get a ready-made loophole which can be turned out in numbers and will probably provide a better shoot than the article hastily built up by the untrained soldier. The plate is not fastened to the box, but slips in between the box and the revetment tray. The screen preserves an unbroken exterior slope to the head-cover and

avoids the black shadowed tunnel which usually vitiates all attempts at concealment.

Overhead Cover, With Curved Iron Sheets, Suitable for a Redoubt (Fig. 8).—The parados is joined up to the head-cover and acts as a reserve parapet in the event of damage to the light blindage, and as a second tier of fire at night.

An Example of Overhead Cover in Reinforced Concrete (Fig. 9). Consists of a 6 by 6-foot gallery with 4 feet roofed in. Steel loophole plates to open forwards and up. (Details of the loopholes were also shown.) An upper platform provides for over-all fire. The hollow under this platform provides full length sleeping space.

FIG. 7.—Box Loophole.



A "shock absorber" of loose boulders is given in front of the interior slope revetment. The soft parados behind is arranged to act as a reserve parapet. The cost of construction is about £9 per rifle. Time, about ten working days with European facilities. An extension of this design by thickening the floor to 3 feet will provide good underground shelter.

Parapet Shelter, German (Fig. 10).—Shows two shelters, for five men each, between traverses 10 meters apart, 3 feet 3 inches head room; 22 inches sitting space each man. Wooden flaps are provided extra to guard against splinters and rain. No interruption to the firing line. No traverses showing above the parapet, and an easy curve round their ends. Parapet shelters of this nature figured prominently in the defences of Kimberley.*

*R. E. Professional Papers, 1900.

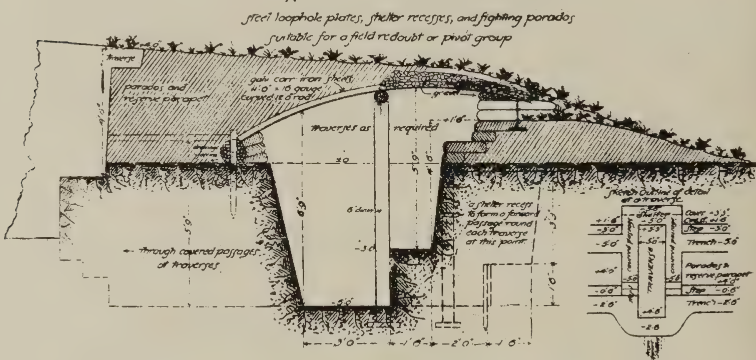
Parapet Shelters, Russian (Fig. 11).—1. Direct support from firing step. 2. Shelter at the lower level, and stronger. Interruption of banquette is not necessary. Men can climb over the earth step, if left, as in the German type (Fig. 10).

Larger Shelters, German (Fig. 12).—For twenty men each. The chief feature of interest is the use of curved steel sheets, of special make, 1 meter wide, as an article of store and bolted together at the crown, in pairs. Suitable for permanent centering to concrete also. Very rapidly used as compared with timber, and more portable.

Sketches for a Dressing Station for Wounded (Fig. 13).—Requirements of dressing station are 2 feet clear space all round a table 6 feet 6 inches by 2 feet 6 inches; a shelf and a seat.

Shelters for supports should enable men to sleep lying down. Seats, if provided, must not encroach on the minimum floor space

FIG. 8.—Type of Fire Trench with Overhead Cover.



or they will probably be demolished after the first night. Each shelter should be numbered, and in direct communication with firing points similarly numbered. The roof is often provided with a "burster layer" of boulders, or better, inclined rails to induce ricochet. The size of shelters is nearly always controlled by the materials available for roofing, and can not be standardized, but the rule is "many and small." As regards actual resistance to shell fire the Japanese consider 4 feet of earth, with a burster layer of boulders, a fair protection against single field gun shells, and 6 or 7 feet against single light field howitzer shells. The Russian figures run from 4 feet 8 inches to 7 feet, and the German from 3 feet 3 inches to 13 feet, while against heavier howitzers (6-inch and upwards) a thickness of 16 to 20 feet of earth or 9 feet of concrete may be necessary.

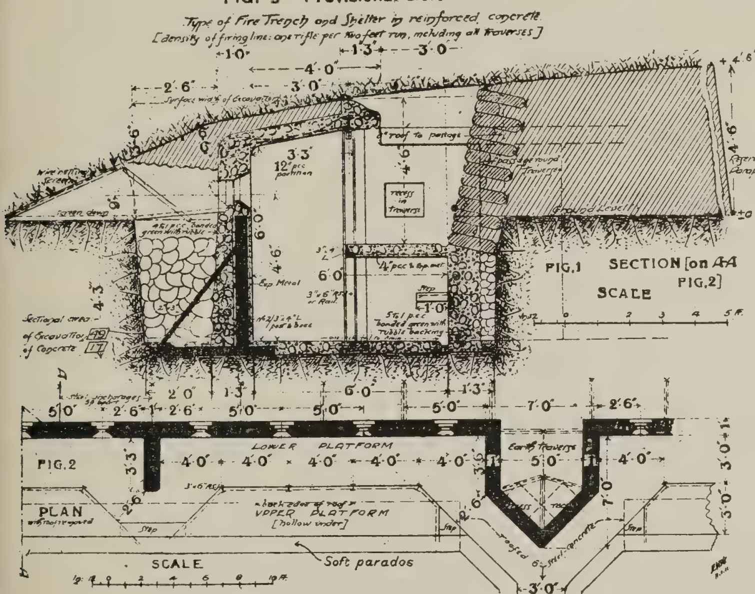
COMMUNICATION EARTHWORKS.

A better realization of the paramount importance of mobility in defence, the occupation of wide frontages, and the increased accu-

racy of artillery fire, have sent up the mileage of communication trenches to a length which may well appal the engineer. Not only safe paths for foot traffic, but sunken ways for pack transport, and sunken roads for wheeled artillery were provided on a prodigious scale by both sides in Manchuria.

Although such communications can not all be developed at once, the safest routes can and must be selected and marked out immediately, so that they can be followed by night or day. Cover from view is the keynote. No one will waste shot upon communications if they do not know when they are in use; but on forward slopes, such as between supports and firing line, communications must

FIG. 9 —Provisional Defences.



either be absolutely concealed or omitted altogether if the firing points are not to be betrayed.

For foot traffic the Russians say "keep the width down to the minimum, about 2 feet 4 inches at the bottom, and make passing places of double width every dozen paces, with occasional recess refuges if there is much traffic. Throw all the earth on the exposed side, and only divide it if doubtful. Go on gradually deepening up to 7 feet as long as the trenches are in use."

In Japan and America experiments in the rapid preparation of long lengths of trench for easy excavation have recently been made by detonating light charges of explosive along the trace.

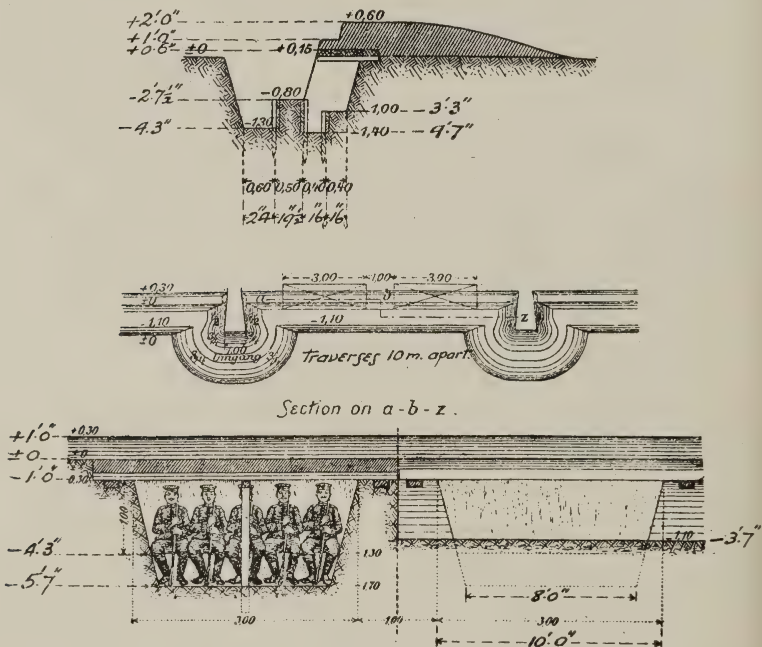
In America, charges of 8 ounces of dynamite dropped into holes jumped about 2 feet apart and 2 feet deep were fired simultaneous-

ly over a considerable length by the shock of detonation of a single charge toward the center. These loosened the ground to a depth of some 4 feet.

In Japan, a continuous charge of 11 ounces per foot run laid on the surface of the ground rendered shovelling easy for some 3 feet below ground.

The immense importance of good covered communications has hitherto been somewhat overshadowed by the supposed paramount importance of early preparation of the fire positions themselves. In peace practices it is quite the exception to find that the neces-

FIG. 10.—Fire Trench with Splinter-Proof Shelters under the Parapet (German).



sary communications within an entrenched zone have even been marked out by the time the fire positions are completed; whereas the process should be exactly reversed. The fire trenches are a matter of hours and, paradox though it may be, the later they are sited and made the more use they are likely to be, whereas communications take weeks to develop sufficiently to ensure that the defenders may compensate for their lack of numbers by their powers of rapid concentration for the production of a superiority of fire at any given point.

Take a visionary example: A stronghold intersected by tube-tunnel railways could be provided by this one means with bomb-proof shelter, countermine galleries, stairs to the surface close in

rear of every good rifle fire position, sidings to run out the truck-mounted guns to every good artillery position, lifts at every point suitable for electric searchlights, and safe signal services, all at a cost less than that of a chain of massive forts and fixed armored batteries. Yet without a single rifle pit it might well be considered likely to prove a harder nut to crack *in the hands of good troops* than any fortress recorded in military history.

OBSTACLES.

Obstacles, like other fieldworks, are good servants to the rifleman

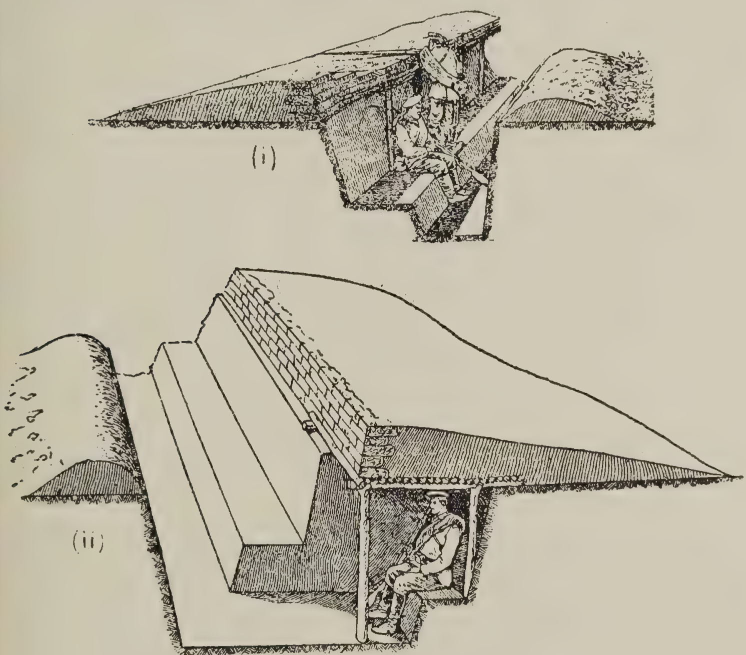


Fig. 11. Splinter-proof shelter under parapet (Russian).

only when they enable him to increase his bag. The German axiom, That the most effective obstacle is the fire of the defender, is only another way of saying that no one is going to try to pass a good obstacle by daylight as long as every foot of it is under effective fire; for the Germans make immensely strong bird-cage wire obstacles themselves.

To ensure the repulse of a heavy night attack not only is a good obstacle essential, especially on short range European battlefields, but is must be, as far as possible, under easy enfilade fire from points definitely prepared and screened from direct fire. Radial fire is weaker than ever in the dark, and as a mere passive obstruction, not controlled by fire, an obstacle soon melts away. The

tendency now is to realize that the night obstacle problem must be divorced from the daylight fire problem, and that it is usually unsound to make the trace of obstacles slavishly conform to the dispositions for general fire effect. In dealing with obstacles and close fighting we are in many points carried back in a moment to the conditions prevailing in the days of Vauban. This is not new, for General Brialmont exemplified it many years ago in his triangular ditch round a fort of oval fire trace. We have now passed beyond the stage of the oval fort in the shooting department and the obstacle question becomes more and more a detached problem. Although it takes many hours to complete an effective obstacle, a very short time will suffice to get a few strands of wire up throughout the trace, and this should be the first move. The finished obstacle also need not be of equal density throughout, but should increase in passive strength in proportion to the difficulty of effective fire control, which may be due either to distance or bad ground. It is usually quicker to "landscape garden" the obstacle itself than to dispose and conceal the earth thrown from a glacis trench in which to sink it. In war we may have to use silver-bright barbed wire picked up locally, but it would be unpardonable to issue it from store until it had been through the green or khaki paint. The Germans issue special instructions for the daubing with clay of all freshly cut wooden surfaces on the posts of wire obstacles, and advocate concealment by transplanted vegetation.

Most obstacles made in peace time convey the impression that the enemy may try to jump them in marching order. Crawling is the pace and attitude of war, and density of wiring near the ground, and a few cartloads of broken glass, are disappointing to crawlers.

No more suitable article of store for use in the rapid construction of obstacles has appeared in recent years than the expanded metal gabion. A continuous central core of such gabions threaded on two or three uncuttable wire ropes and densely wired at all heights to other gabions spaced out instead of posts, the whole stiffened with a few steel fencing posts as holdfasts, and well strewn with broken bottles lightly earthed over, should afford plenty of promotion for the enemy's sappers.

OBSERVATION AND FIRE CONTROL IN ENTRENCHMENTS.

There is no necessity to emphasize the important part now played in defence by protected look-out and fire control posts. Several examples of infantry observation posts have already appeared in the plans of works shown, and it will have been noted that they are sited quite independently of the firing portions of the works. The generally agreed principles for their disposition are that there should be enough look-out posts to keep the whole field of fire and every possible line of approach under close observation by day. If possible, not more than about 45° of arc

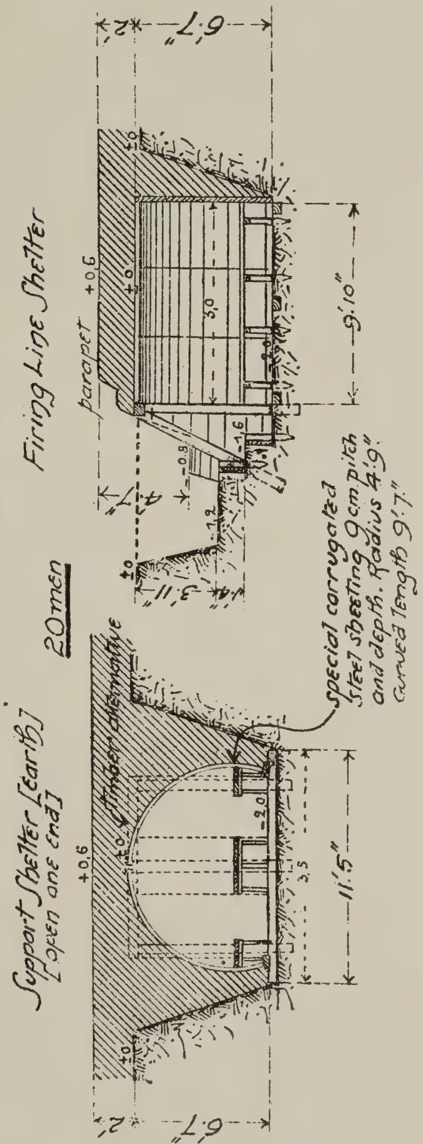


FIG. 12.—Types of Shelters (German).

should be allotted to one observer. The German normal type shows about 40°, the Russian 75°, which appears to be rather too much.

Since the observers must remain on duty under artillery fire, storm and tempest, protection and concealment are of paramount importance. They are therefore best well away from visible entrenchments, but must be in communication with the fire-control posts both by signal and by covered walk, and if in advance must be well screened from reverse fire. By night a different set of look-out posts would usually be required if the defences included obstacles, and the men in such posts should be close to any alarm guns or flare lights provided, and able to operate them at will independently from any involuntary operation of them by the enemy.

An officers' fire-control post would usually include a look-out pit, with a splinter-proof shelter nearby for the telephone, signallers, and orderlies awaiting messages. This look-out pit requires a wide arc of view and should include all ground under fire by the men under the officer's command. An ordinary fire loophole is quite unsuitable. A horizontal slit, in steel plate if possible, about 2 inches high, 12 inches wide and 5 feet above the observing step, is usually convenient. A seat should be provided 2 feet 1 inch below, and an elbow shelf (12 inches wide) 1 foot 1 inch below the sill of the look-out slit for convenience in the use of binoculars.

In the O. C.'s post a megaphone aimed at one or two of the adjacent subordinate fire-control posts should be kept fixed and ready for use at night, in addition to a loose megaphone on a peg. Rockets and light signals for communicating at night with more distant posts, or with artillery, may be stored in the O. C.'s shelter. The map (ruled in the same squares as that with the artillery) should be oriented, in a good light, but protected from weather. Enlarged sketches of foreground can ignore the relative sizes of the conventional signs and should have the important range features boldly indicated and lettered to read easily *when the reader is facing the feature,* not, as sometimes seen, written upside down from a mechanical sense of homage to "α Ursæ Minoris." In addition to the lists of ranges chalked on the sides of traverses, etc., every range so marked should be actually verified by rifle fire—a precaution often overlooked—and every man should constantly be made to repeat from memory the ranges to his targets, even when a certain number of them are already marked on the ground with their actual range by a number of visible objects corresponding to the number of hundreds of yards in the distance.

COVER FOR MACHINE AND FIELD GUNS.

Machine Guns.—All the principal armies now fight their machine guns from low tripod or sledge mountings—the latter being favored by Germany and Russia.

Our "Maxim" and the French "Puteaux" fire over a height of from 14½ to 30 inches. The German sledge mounting fires from a ramp varying from 4 inches to 2 feet 4 inches below the fire crest (Fig. 15), and the Russian variety (Fig. 16) fires off a level shelf some 3 feet wide and 7 inches below the fire crest. The advantages of the shallow sledge mounting working on an inclined ramp which permits of instant withdrawal are obvious.

Cover for the detachments and a protected look-out must be very near and very safe, since the weapon is essentially of the "now or never" variety and the detachments have to keep alert

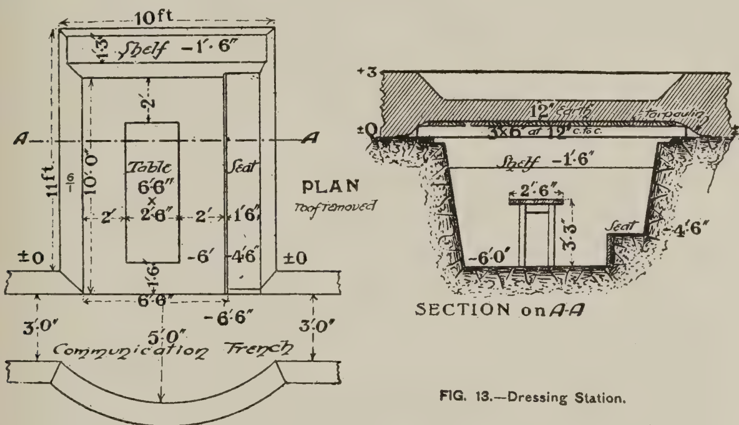


FIG. 13.—Dressing Station.

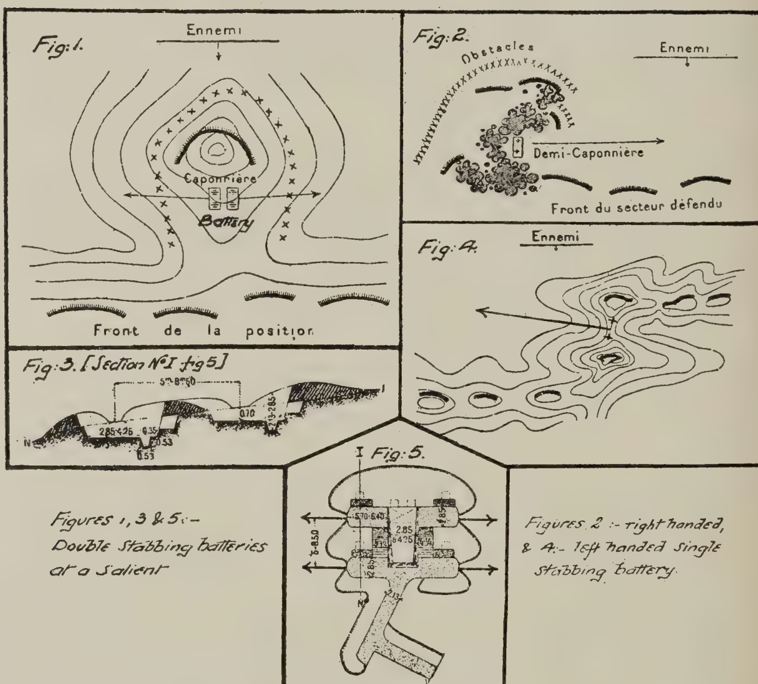
under cover for hours or even days ready for a few seconds' shooting at the warning of the look-out man (Fig. 15).

Stabbing batteries of Machine or Field Guns (Fig. 14) for close range flank fire in the open field are greatly favored by the Russians, and find a place in all recent designs for permanent works. In the latter they are usually found in an armored gorge casemate caponiere, and in fieldworks behind some natural salient screen in front of the general fire line. Their sudden leap into prominence is no doubt partly due to the success at Port Arthur of the famous "Open Caponiere No. 3" (Hachi-maki-Yama), now known as "G," where a couple of field guns and a couple of machine guns thus mounted gave such trouble to the Japanese that they were compelled, for the first time in the history of war, to use 500-pound shells from 11-inch howitzers against simple earthworks.

Cover for Field Guns.—The chief tendencies in recent types of artificial cover for guns are in the direction of concealment, better flank traversing from oblique fire, and blinded cover for detach-

ments and ammunition. The formal battery is as extinct as the formal fire trench, and a good hidden shoot with the maximum of protection are the ruling factors. A favorite opening move with the Japanese field artillery seems to have been the standing up of half-a-dozen large sacks filled with earth (as gabions) to act as revetment to detachment shelter parapets on either side of the gun. In spite of its minor disadvantages the sunken gun pit, without visible parapet, is the normal emplacement, and separate

FIG. 14.—Stabbing Batteries (Russia).



screens to defilade flash and hide the gun shields are commonly provided. Both Russians and Japanese agree as to the importance of providing separate emplacements for night and day use, and spare emplacements in the rifle lines to enable guns to join in the shooting at assaulting mobs. The latter require gun shelters and easy ramps not unlike the types of thirty years ago.

German Field Gun Emplacements (Fig. 17).—Battery without parapet to fire over ground level, sunk 2 feet 8 inches at gun emplacements and 5 feet 7 inches at shelters. Similar to the gun pits used by us for 4.7-inch guns at Modder River Camp in 1899.

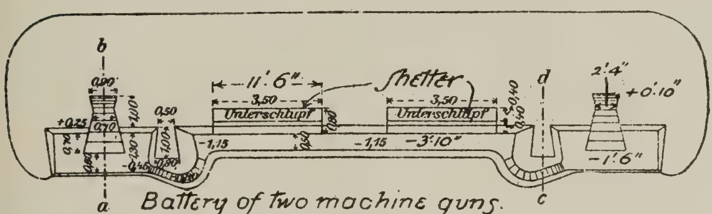
Russian Field Gun and Howitzer Emplacements (Fig. 18).—The

types of Russian field gun and howitzer emplacements are principally of interest from the practical simplicity of their trace and the extent to which blinded cover is illustrated. The illustration of cover for an ammunition limber or wagon (Fig. 18, 1) close to the gun is of special interest since so many armies have left this point to the long list of brilliant improvisations expected at short notice on the battlefield.

SUMMARY.

Time will not permit of any examination of the many recent

FIG. 15.—Types of Machine Gun Emplacements (Germany)



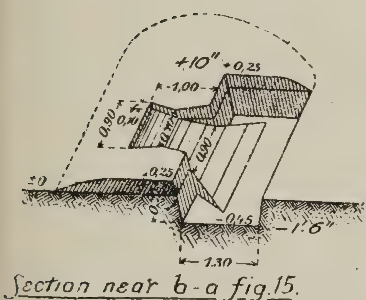
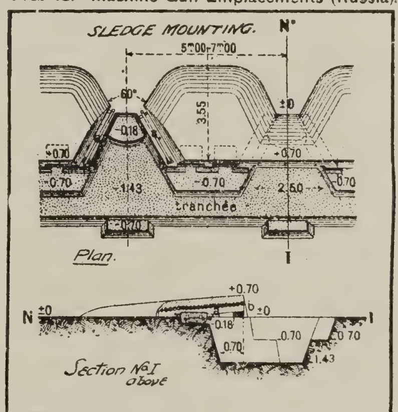


FIG. 16.—Machine Gun Emplacements (Russia).



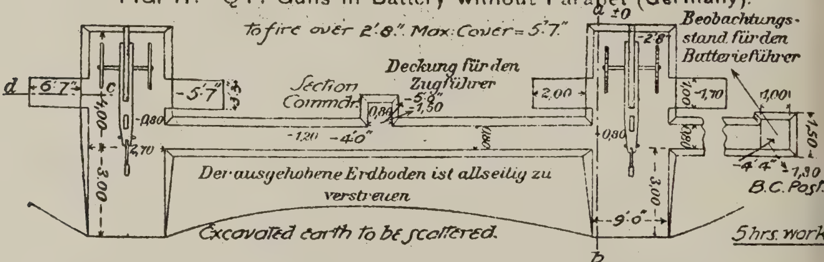
developments of interest in connection with other branches of field engineering, such as searchlights and other field illuminations, land mines and explosives, field tramways, telephones, alarm signals, tools and materials, bridging, and numerous subjects of equal interest, and it only remains briefly to sum up our own situation with regard to field engineering to-day.

If in the course of the examination of a few recent types of works there may have been an apparent tendency to dwell unduly upon the material means of defence or attack, it must not be thought that the paramount human element, or the spirit of the man who is "out to win," have been overlooked or ignored.

We know well that the best instances of good defences in history owed but little to the skill of the engineer as compared with the valor of the man behind the gun, yet it is still the duty of the engineer to apply his skill in seeking for strength, and still further refinements of strength, and to place them at the disposal of the man who does the shooting — to use or not to use — as he may think best for the occasion. The strength of field engineering, like the paint on the artist's canvas, produces nothing of value unless it is applied in the right place, in the right quantity, at the right time, and of the proper form and color. Otherwise it remains, as the Malay idiom expresses it, "of no more use than a shield in a pawnshop."

Whatever critics may say to the contrary, the defences constructed by the Russian engineers around Port Arthur were immensely strong; far stronger than their own constructors thought them to be; but they were not in harmony with the spirit of men who are determined to conquer, not merely to baffle their enemy.

FIG. 17. Q F. Guns in Battery without Parapet (Germany).



The science of field engineering applied in the true spirit appears to aim at supplying the means for that economy of force which enables the skilled jiu-jitsu wrestler to divert the ill-balanced strength of the impetuous giant in such a way as to hasten his downfall.

The field engineer must, however, be careful that it is the enemy, and not his own side, that he arranges to mystify, mislead, and surprise; and all his schemes should have that stamp of effective simplicity which will provide the maximum of opportunity for the *average* soldier, led with *average* skill, to inflict the maximum of loss upon the enemy. The good fighting man, now, as always, can make the best of his opportunities from moment to moment, though the best of soldiers can not to-day ignore the spade and win through with rifle alone. It must always be remembered that it is the duty of the engineer to provide the soldier of the line with the assistance he *wants*—not something which the engineer thinks he *ought* to want, and is lucky to get. It is not enough for the engineer to set the stage and have his own lines by heart, he must rehearse the play from the very beginning with the gunner and rifleman present on the stage in full costume, or he may find at the

first public performance, like the Austrian engineers at Königgrätz, that he has forgotten to inform the rest of the caste where the performance is supposed to take place, and where the properties are kept.

In this country we are, perhaps, fortunate in escaping the fatal magnetic attraction of cramped and obsolete types of land forts, which so often throughout military history have administered the slow poison of defeat to those who put their trust in

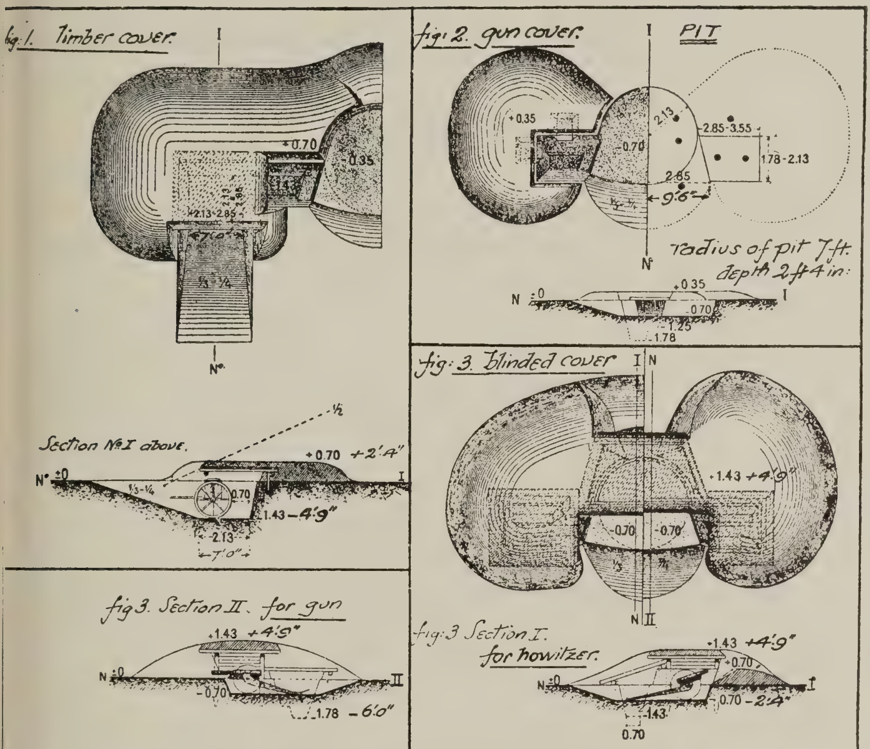


Fig. 18. Types of field artillery cover (Russia).

them. But for this very reason it is all the more necessary for us to be fully prepared with matured plans for the rapid and intelligent anticipation of events, and for the use of provisional defences applied so skilfully that, however quickly the decisive point for using the leverage of one of our striking forces may be moved, that lever (the striking force) may always find a strong and tactically opportune fulcrum against which to bear at the decisive moment.

It is abundantly clear that, as fighting goes to-day, it is possible to extemporize effective fieldworks both from the smallest beginnings and in a comparatively short time; and that such works, if used so as to develop to the full the power of the modern

firearm, possess enough of the passive strength of permanent fortress works for resistance, while they are of far greater value for making good in the wake of the offensive, owing to their being strategically sounder and tactically more opportune. But such works can be carried out *only by engineers trained for this purpose.*

The master thought in all education in the use of the spade should not be that men well entrenched can *resist* the attack of several times their number, but that men who are trained and equipped to use the spade so as to increase the power of their other weapons should be able to surround and destroy or capture a larger force which has not this advantage.

Erratum

In connection with the article by Lieut. James G. Steese on "The Corps of Engineers and the Isthmian Canal," which appeared in the July-August number of the PROFESSIONAL MEMOIRS, the following correction should be made:

Col. Wm. M. Black (then Major) and Lieut. Mark Brooke, Corps of Engineers, were placed under the orders of the Isthmian Canal Commission on April 1, 1903, instead of March, 1904. They proceeded to Panama and were stationed at Culebra. The duty assigned to Colonel Black was to observe and report on the work of the new Panama Canal Company, reporting particularly on the expenditures made month by month and the work accomplished. Colonel Black was absent from the Isthmus May 4, 1904, when the actual transfer was made, but returned thereto a few days after and remained as Acting Chief Engineer of the Commission until the arrival of Mr. John F. Wallace, who took charge of the work July 1, 1904.

Selected Articles of Engineering Interest

Compiled by Henry E. Haferkorn, Librarian, Engineer School.

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.

(2) American Machinist.

(3) Canadian Engineer.

(4) Canadian Soc. of Engineers. Trans.

(5) Cassier's Magazine.

(6) Cement.

(7) Cement Age.

(8) Cornell Civil Engineer.

(9) Electrical Review (London).

(10) Engineer (London).

(11) Engineering (London).

(12) Engineering-Contracting.

(13) Engineering Magazine.

(14) Engineering News.

(15) Engineering Record.

(16) De Ingenieur (Hague, Holland).

(17) Journal of American Society of Mechanical Engineers.

(18) Journal of Western Society of Engineers.

(19) Journal of Franklin Institute.

(20) Journal of Royal United Service Institution (London).

(21) Proceedings, American Society of Civil Engineers.

(22) Proceedings, Engineers' Club of Philadelphia.

(23) Municipal Engineering.

(24) Municipal Journal and Engineer.

(25) Railway Age Gazette.

(26) Revue Generale des Chemins de Fer (Paris).

(27) Scientific American.

(28) Scientific American Supplement.

(29) Transactions, American Society of Civil Engineers.

(30) Professional Memoirs, Corps of Engineers.

(31) Journal of the Royal Artillery (Woolwich, England).

(32) Royal Engineers' Journal (Chatham, England).

(33) Proceedings Brooklyn Engineers' Club.

(34) Concrete.

(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).

(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden)

(37) Revue d'Artillerie (Paris).

(38) Kriegstechnische Zeitschrift (Berlin).

(39) The Contractor.

(40 Cement Era.

(41) Canal Record (Ancon, C. Z.).

(42) Proceedings, Engineers' Society of Western Pennsylvania.

(43) Journal, United States Artillery.

(44) Transactions, Society of Engineers (London).

(45) Journal, Association of Engineering Societies.

(46) United States Naval Institute. Proceedings.

(47) Revue du Genie Militaire (Paris).

(48) La Technique Moderne (Paris).

(49) Electrical World.

(50) Electrical Review (Chicago).

(51) Journal, Military Service Institution

(52) Barge Canal Bulletin.

(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)

(70) Minutes of Proceedings, Institute of Civil Engineers, London.

BANK PROTECTION.

Grass soil binders for protecting embankments from erosion. (14), July 11, 1912.—The sea defenses at Lowestoft. (15), July 13, 1912.

BARGES.

Cost, life, and repairs of barges, tow-boats, and other floating plant used in the U. S. improvement of the Upper Mississippi River. (12), July 17, 1912.—Ferro-concrete sludge-pumping ponton; Manchester ship canal. W. N. Twelvetrees. (11), June 14, 1912. D. I.—in: River and harbor notes from foreign lands. F. B. Downing. (30), Sept.-Oct., 1912. I.

BREAKWATERS.

The Colombo breakwater. (15), July 6, 1912.—Description and cost of concrete superstructures for breakwaters at Harbor Beach, Mich. E. J. Duffies. (30), Sept.-Oct., 1912. D.—Effect of storms on a lake breakwater. L. W. Goddard. (14), July 18, 1912. I.—Methods and cost of constructing stone-filled timber breakwater, Lincoln Park extension, Chicago, Ill. (12), June 19, 1912. D.—More recent types of breakwater construction and the unit cost of several of importance in the United States. (12), June 26, 1912. D.

CANAL BOATS.

Factors affecting the safe and economical operation of boats in a restricted channel in the Hudson River. R. D. Black; W. P. Benjamin. (30), Sept.-Oct., 1912. D.

CANAL LIFTS.

See Hydraulic power. J. Horner. (5), June, 1912. D. I.

CANALS.

Canadian canals. (11), July 26, 1912.—Dimensions for canals of heavy traffic; a summary of practice in leading countries. (12), July 24, 1912.—Enlargement of the Kaiser Wilhelm Canal. (11), June 21, 1912.—Huge canal project. (10), June 7, 1912.—Milan-Venice Canal. (10), July 5, 1912.—Refection des maconneries du souterrain de mauvages sur le canal de la Marne au Rhin. M. F. Launay. (1), May-June, 1912. D.—Water supply of the Orleans Canal (France) by the elevation of water from pool to pool. M. Rousseau. Tr. C. H. Brown. (30), Sept.-Oct., 1912. D.

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Further investigations of Puzzolan-Portland cements. E. Duryee. (14), Aug. 15, 1912.—Government standard specifications for Portland cement. (7), June, 1912.

COAST DEFENSE.

Defense of the German coasts. (20), June, 1912. D.

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Straightening a cofferdam. (39), Aug. 1, 1912. D.—see: Difficult substructure work on the United States barge office, New York. (15), Aug. 3, 1912. I.—Hydro-electric project of the Mississippi River Power Co. at Keokuk, Ia. (12), Aug. 14, 21, 1912. D. I.—New canalized channels in the Detroit River. J. C. Mills. (5), July, 1912. D. I.—Water power development in the South. J. A. Switzer. (5), July, 1912. D. I.

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Concrete in sea water. J. S. Sewell. (14), July 18, 1912.—European studies of concretes resistant to destruction by sea water. (12), Aug. 21, 1912.—Destruction of concrete between tides in sea water. W. B. Mackenzie. (14), July 4, 1912. I.—Lumber dock at Balboa, Canal Zone. G. R. Goethals. (15), July 20, 1912. D. I.—Method and cost of constructing a concrete and steel ore dock for the Duluth and Iron Range R. R. L. Clapper. (12), July 17, 1912. D. I.—Notes on methods of securing impermeable concrete for marine works. (12), Aug. 21, 1912.—Oil-mixed concrete. R. G. Keevill. (11), June 7, 1912.—Preservation of reinforced concrete in sea water. E. Burr. (12), July 31, 1912.—Reinforced concrete lumber dock at Balboa. W. J. Spalding. (34), Aug., 1912. D. I.—Ferro-concrete sludge-pumping

ponton: Manchester ship canal. W. N. Twelvetrees. (11), June 14, 1912. D. I.—in: River and harbor notes from foreign lands. F. B. Downing. (30), Sept.-Oct., 1912. I.

DAMS.

Accident at Dam No. 26, Ohio River. (15), Aug. 17, 1912. I.—Arrow Rock Dam, Boise project, U. S. Reclamation Service. (12), Aug. 21, 1912. D.; (39), July 1, 1912. D.—The Austin Dam failure. F. P. McKibben. (45), June, 1912. D. I.—Concrete-faced earth dam at McAlester, Okla. (15), Aug. 3, 1912. D. I.—Construction des barrages reservoirs en Allemagne. (46), Aug. 1, 1912. D.—Construction of Engle Dam. (39), Aug. 15, 1912. D.—Damming the world's greatest river. H. S. Rogers. (28), Aug. 10, 1912. I.—Delta Dam and storage reservoirs for supplying water to the Rome Summit level,New York State barge canal. E. Low. (12), June 19, 1912. D. I.—in: Derwent Valley water works. (10), July 19, 1912. D. I.—Filling cut-off trenches below earth dams. (15), July 13, 1912.—in: Hydro-electric development in western North Carolina. N. Buckner. (5), July, 1912. I.—in: Hydro-electric plant as Estacada, Ore. (49), July 13, 1912. D. I.—in: Hydro-electric project of the Mississippi River Power Co., Keokuk, Ia. (12), Aug. 14, 1912. D. I.; (12), Aug. 21, 1912.—Recent studies of ice pressure and their consideration in masonry dam design. (12), June 26, 1912. D.—Repairing the dam at Hatfield, Wis. (15), July 27, 1912. D. I.—Safe dam construction in New York. J. Moore. (14), June 27, 1912.—Simple formula for the design of masonry reservoir dams. (12), Aug. 7, 1912. D.—Tension in masonry dams. J. C. Trautwine, jr. (12), Aug. 7, 1912.—in: Water power development in South. J. A. Switzer. (5), July, 1912. D. I.

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Demolition by explosives at Port Talbot. (10), June 7, 1912. D. I.—Demolition of wire entanglements by means of explosives. (32), Aug., 1912. D.

DERRICKS.

Combination pile driver and derrick for trestle construction. (12), Aug. 14, 1912. D.—A handy portable derrick. (12), July 10, 1912. I.

DIKES.

Construction of dike wall, using reinforced concrete caissons at the port of Havre, France. (12), Aug. 7, 1912. D.

DOCK MACHINERY.

Cargo loading and discharging appliances at Immingham Dock. (11), June 14, 1912. D. I.—Mechanical appliances on the Panama Canal. J. F. Springer. (5), June, 1912. I.

DOCKS.

Gladstone Dry Dock at Liverpool. (10), June 7, 1912. I.—The Immingham Dock. (10), June 7, 14, 28, 1912. D. I.; (11), July 26, 1912.—Lumber dock at Balboa, Canal Zone. R. G. Goethals. (15), July 20, 1912. D. I.—the same, W. J. Spalding. (34), Aug., 1912. D. I.—Method and cost of constructing a concrete and steel ore dock for the Duluth and Iron Range R. R. L. Clapper. (12), July 17, 1912. D. I.—New naval dry dock at New York. (10), June 7, 1912.

DRAINAGE.

The Little River drainage works in Missouri. W. A. O'Brien. (15), June 22, 1912. D.—Methods used in effecting the preliminary organization of a 450,000-acre drainage and levee project in Southwestern Missouri. L. T. Berthe. (12), July 31, 1912. D.

DREDGES AND DREDGING.

Dredges to excavate 71 feet below sea level. (39), Aug. 15, 1912; (12), Aug. 21, 1912.—Feathering paddle wheels for U. S. self-propelling hydraulic dredges. (14), Aug. 15, 1912. D.—Materiel de dragage de canal. M. Galliot. (1), May-June, 1912.—Modern dredges of the bucket and suction type. F. C. Perkins. (28), June 29, 1912. I.—in: New canalized channels in the Detroit River. J. C. Mills. (5), July,

1912. D. I.—Shifting a dredge overland. (39), Aug. 15, 1912.—Suction dredge on the Panama Canal. (39), Aug. 1, 1912. I.—United States Government contract dredging. (14), July 11, 1912.

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Arbitration clauses in engineering contracts. (15), July 27, 1912.

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Some recent tendencies in field engineering. E. B. H. Wilson. (30), Sept.-Oct., 1912. D.

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Cherry Creek flood, Denver, Colo. C. W. Comstock. (14), Aug. 15, 1912. D. I.—Flood problem of Pittsburgh. K. C. Grant. (10), July 19, 26, 1912. D. I.; (11), Aug. 2, 1912. D. I.—The great Mississippi River flood of 1912. Report. T. G. Dabney. (15), July 6, 1912.—Reservoir control of Mississippi floods. (14), Aug. 1, 1912.—Results of Denver flood. (15), July 27, 1912. I.

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New harbor at Frankfort-on-Maine. (10), June 7, 1912.—in: New graving dock at Belfast. W. R. Kelly. (11), Aug. 2, 1912—Note sur les travaux en cours et en projet au port de Saint-Nazaire. M. E. Epinay. (1), May-June, 1912. D.—River and harbor notes from foreign lands. F. B. Downing. (30), Sept.-Oct., 1912. D.

HYDROELECTRIC PLANTS.

Damming the world's greatest river. H. S. Rogers. (28), Aug. 10, 1912. I.—Hydroelectric development in western North Carolina. N. Buckner. (5), July, 1912. I.—Water power development in the South. J. A. Switzer. (5), June, July, 1912. D. I.—Hydroelectric project of the Mississippi River Power Co. at Keokuk, Ia. (12), Aug. 14, 21, 1912. D. I.—The San Joaquin hydroelectric power installation. F. C. Perkins. (28), July 6, 1912.—Winnipeg hydroelectric power station. (11), July 26, Aug. 2, 1912. D. I.

INLAND NAVIGATION.

Huge waterway schemes. (10), July 5, 1912.—Swiss water transportation. (15), July 13, 1912.—Lesvoies navigables et les projets de canalisation pour l'Allemagne du sud. M. R. Hennig. (1), May-June, 1912. D.

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Can an impermeable core be used in Mississippi levees? J. C. Morris. (14), June 27, 1912.—Foundation weakness in Mississippi levees. T. G. Dabney. (14), Aug. 1, 1912.

LIGHT-HOUSES.

New French lighthouse at Ushaut. (11), July 26, 1912. D. I.

LOCKS AND LOCK GATES.

Accident to gates of lock 22, Welland Canal, near Thorold, Ont. E. Low. (14), July 11, 1912. D. I.—Les ecluses du canal de Panama. L. G. Levy. (48), June 15, 1912. D.; July 1, 1912. D. I.—Features of lock gate construction. (41), July 3, 1912.—Gears for Panama emergency gates. (2), Aug. 22, 1912. D. I.—Mechanical appliances on the Panama Canal. J. F. Springer. (5), June, 1912. I.—Progress at the Gatun Locks, Panama Canal. (14), Aug. 15, 1912. I.—Rapid drills for the Panama Canal. (15), June 22, 1912. I.—Reconstruction of Boulter's lock. (10), June 7, 1912. D. I.—Damming the world's greatest river. H. S. Rogers. (28), Aug. 10, 1912. I.—Hydraulic power. J. Horner. (5), June, 1912. D. I.—Water power development in the South. J. A. Switzer. (5), July, 1912. D. I.

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Il nuovo materiale da pont nell' esercito tedesco. (Rivista di artiglieria e genio), May, 1912. D.Passage of ponton bridges by mechanical transport. I. C. Bearne. (51), July-Aug., 1912. I.—A portable field girder, and a simple method of launching. R. L. McClintock. (32), July, 1912. D. I.

PANAMA CANAL.

Gears for Panama emergency gates. (2), Aug. 22, 1912. D. I.—Latest news from the Culebra cut. (15), Aug. 3, 1912.—Lumber dock at Balboa, Canal Zone. R. G. Goethals. (15), July 20, 1912. D. I.—Mechanical appliances on the Panama Canal. J. F. Springer. (5), June, 1912. I.—Miraflores spillway of the Panama Canal. E. C. Sherman. (15), Aug. 3, 1912. D.—Progress at the Gatun locks, Panama canal. (14), Aug. 15, 1912. D. I.—Suction dredge on the Panama Canal. (39), August 1, 1912. I.—Summary of eight years work on the Panama Canal. (14), June 27, 1912.

PIERS.

A pier built on cylinders cast as concrete shells and filled in place. H. L. Muchemore. (15), June 22, 1912. I.—River Wear south protective pier. (10), June 7, 1912.

PILES AND PILE DRIVING.

Combination pile driver and derrick for trestle construction. (12), Aug. 14, 1912. D.—in: Difficult substructure work on the United States barge office, New York. (15), Aug. 3, 1912. I.

POLLUTION OF STREAMS.

Pollution of a river by placer mining. (15), Aug. 3, 1912. I.—Water pollution control in Ohio. (14), July 11, 1912.—West Riding Rivers and trade effluents. (11), June 14, July 26, 1912.

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Improvement of the Neponset River in Massachusetts. E. M. Blake. (14), June 27, 1912.—Methods and costs of making the United States improvements at Coon Slough on the Upper Mississippi River. C. W. Durham. (12), July 31, 1912. D.—Methods of river improvement by regulation and dredging. W. H. Harts. (12), July 24, 1912.—New canalized channels in the Detroit River. J. C. Mills. (5) July, 1912. D. I.—River and harbor notes from foreign lands. F. B. Downing. (30), Sept.-Oct., 1912. D.—Method of plotting river discharge data. E. W. Schoder. (15), Aug. 3, 1912. D.

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Plan for the regulation of the Platte River. (15), July 27, 1912.—Turning the Black River back into its former bed. (15), Aug. 3, 1912. D. I.—Notes on the regulation of the river Nile. A. W. Robinson. (4), Transactions, v. 25, pt. 2, Oct.-Dec., 1911.

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Methods and costs of rock excavation in the harbors of San Esteban de Dravia and Port de Bilboa, Spain. (12), June 19, 1912. D.—Methods of excavating subaqueous hard rock in the Trollhattan Canal, Sweden. (12), July 3, 1912.—Scientific management of the rock drill. (5), Supplement, June, 1912.—in: New canalized channels in the Detroit River. J. C. Mills. (5), July, 1912. D. I.

SEARCHLIGHTS.

Defense light mirrors. R. Law. (Commonwealth Military Journal), June, 1912. D.—Searchlights. J. M. Heslop. (5), June, 1912. D. I.

SEA WALLS.

Sea wall of rock and reinforced concrete construction. (15), Aug. 10, 1912.

THERMIT WELDING.

Methods and cost of repairing dredge park in the field by thermit and oxy-acetylene welding. (12), July 17, 1912.

WAVE ACTION.

Influence of navigation on movement of water in canals and upon the erosion of the canal bed. (In Dutch language), (16), April 14, 1912.—The tidal wave and current. J. F. Ruthven. (20), July, 1912.

Early Experience with Balloons in War

BY

Brig. Gen. HENRY L. ABBOT
United States Army, Retired

The recent rapid development of flying machines in the form of dirigibles and aeroplanes, and their prospective uses in future wars by land and sea, naturally give interest to records of the part their progenitor, the spherical balloon, played in our Civil War.

Aerial flight was then in its infancy. At Versailles in September, 1783, Montgolfier sent up a hot-air balloon carrying as passengers a sheep, a duck, and a cock to a height exceeding 1,000 feet; and the animals descended without accident. The first person to make an ascension was M. de Rozier, who rose in a balloon of the hot-air type to a height of 500 feet over Paris in November, 1783, and descended in safety; but in a later attempt to cross the English Channel in a balloon he lost his life. Hydrogen and, still later, coal gas soon took the place of hot-air as the elevating agent, and ascensions became more frequent. The American pioneer in this new scientific field was Wise, who made his first flight in May, 1835, using hydrogen gas.

So far as I have been able to learn, no serious test of the military efficiency of spherical balloons, the only type then known, had been made at the date of the early campaigns in Virginia in 1861-1862, although the idea of experimenting with them had been entertained in some of the French Revolution and Napoleonic wars and in our war with Mexico. Thus in August, 1797, General Hoche wrote to the Directory: ''I beg to inform you that the Army on the Sambre and Meuse has a company of balloonists, for which it can find no use; perhaps it would be better to let it join the 17th Military Division, where it would be nearer the capital, and so in a better position to do useful work.'' The field was practically undeveloped then; but in 1870-1871, shortly after our war, they were extensively used by the French in the siege of Paris to open communication with the exterior—one successfully carrying Gambetta

to organize a fresh army in the provinces. The Germans made little or no use of them.

Their record in our Civil War was the following: Early in 1861, one of the first three-months' volunteer regiments from Rhode Island brought with them to Washington two balloons. They were the property of James Allen, of Providence, who offered them to the Governor of his State in the belief that they would be of service, and who himself accompanied them. They were finally placed at Camp McDowell, on the right bank of the Potomac River near one of our advanced positions at Falls Church, and were at first, I think, under the provisional supervision of Dr. Helme. Reporting to General McDowell on July 5, 1861, as one of the Engineer officers of his staff, he at once ordered me to take charge of these balloons and determine by actual trial whether they would be of practical utility in the advance soon to be made. I went to their location, and found that one of them was an old cotton affair which had long been in use, while the other was made of silk and was in good condition. They were accompanied by a crude apparatus for generating hydrogen by the action of dilute sulphuric acid upon iron turnings, but in spite of all their efforts the operators had only succeeded in about half filling the silk one. By working all day and night we succeeded in accumulating sufficient gas to make an ascension to a height of about 500 feet, but the wind was blowing so strongly that the operators did not dare to pay out more rope and let me rise higher, a length of about 1,000 feet being available. I at once began observations and found the results rather disappointing. The view commanded a wide range of country undulating in character, with much wooded land and interspersed open fields, but the motion of the balloon caused by the wind was so great that it was impossible to keep my field telescope directed upon a single object long enough to study its character, and even with my smaller binocular I doubted whether it would be possible to count the number of guns in a battery at any considerable range. Furthermore, the trees so masked the fields behind them that no close estimate of the number of troops present could well be made. The advance was soon to begin; and I reported to General McDowell that no dependence could be made on the gas-generating apparatus, and that the only available manner to utilize the balloons in the movement would be to have them filled with coal gas at Alexandria and conducted by men holding guide ropes, using one balloon as a supply reservoir for the other. He

approved this plan, and ordered me to make the needful preparations.

The two balloons were transported empty to Alexandria, and with the assistance of the depot quartermaster, Captain Tyler, arrangements were made to have them filled at early dawn on July 14, that hour being chosen as probably the least likely to be interrupted by wind. A detail of sixty men was obtained from, I think, the New York (Ellsworth) Zouave Regiment to conduct them when filled to Falls Church, with a view to having them accompany the advance of Gen. Daniel Tyler's Division when time for the movement had arrived. The old cotton balloon was filled first; but no sooner was it conducted to one side than a loud puff was heard, and its fragments fell to the ground. The quartermaster had taken great interest in the operation and was standing by my side; he remarked significantly: "Abbot, I did not join the Army to be a bird." The other balloon was successfully filled; and the detail of sixty men started with it up the Washington pike, intending to follow that road to the point where the branch to Falls Church diverges, and then to follow that. The wind began to rise with the sun, and the sixty men in their brilliant uniform executed vigorous calisthenics as the gusts came. We worried along nearly to the point where our branch road diverged when suddenly a furious gust occurred. The detail, struggling and shouting, was slowly pulled toward the river in spite of their efforts until the balloon in one of its stately plunges struck a telegraph pole. There was a puff of gas, and our work was ended. I rode back to Arlington, not sorry to be rid of what I had become convinced was destined to certain failure, and General McDowell remarked that he was glad to be relieved from an experiment for which the means provided were so inadequate.

At Washington, during the autumn of 1861, and in the Peninsular Campaign of 1862, the utility of captive balloons was put to a more elaborate test. An aeronaut of note, Mr. T. S. C. Lowe, was called to Washington and an equipment of a dozen or more balloons was provided, together with an efficient plant for generating hydrogen gas. Mr. James Allen also did good service with them. During the siege of Yorktown, and on the banks of the Chickahominy, they were useful in the way of reconnaissances, the available height being limited to about a thousand feet. Gen. Fitz John Porter and many other officers made good use of them; on one occasion he had a remarkable experience: the rope by which

he was anchored parted, and rising to a considerable height he was swept over the defences of Yorktown; he had the presence of mind to remember that the surface stratum of air was blowing in the opposite direction, and by operating the valve he sunk back to it, and thus returned uncaptured to our lines, having carefully inspected the hostile works. It is believed that this is the first military aerial flight on record.

Seeing our balloons often in the air at Yorktown and on the lines of the Chickahominy, the Confederates desired to have some of their own. The blockade rendered the importation of materials extremely difficult, and an appeal was made to the ladies, who patriotically sacrificed their silk dresses. A single balloon was manufactured, and, after being filled with illuminating gas at the works at Richmond, was towed by a locomotive on the York River Railroad to the vicinity of the point we were occupying at that time near Harrisons Landing. Some use was made of it under the direction, I believe, of Gen. E. P. Alexander; but finally, when operating from a steamboat which ran aground on James River, this, the only Confederate balloon, fell into the hand of our Navy.

The question of transportation was a difficult one, and when the Army of the Potomac was transferred back to the vicinity of Washington balloons played a less important part in its operations. They were used at Fredericksburg with some success and were transported to Chancellorsville, but after the battle they disappeared and were never used in the later operations of either army. General Burnside had taken a balloon with his expedition to Roanoke Island; and one was used on the Mississippi at the attack on Island No. 10 where, as was reported, it did good service in directing the artillery fire. Elsewhere no use of them was made by the Western Armies during the war.

Our experience demonstrated that the device of captive balloons possessed decided war merit; and, furthermore, that it annoyed the enemy and made them believe they were laboring under a serious disadvantage in not having them; but the difficulties of transportation, including the bulky apparatus for generating the hydrogen, and the absence of any regular system for administration and for the prompt communication of the results of observation to the commanding generals, excluded them from use during the later campaigns of the war. Modern improvements, rendering possible rapid aerial flights and prompt communication of the results by aerial telegraphy, to say nothing of photography, will go far to increase the value of this new arm of service; but, on the other hand, possible aerial conflicts must be anticipated. Recent Italian experience in Tripoli has thrown much light on the advantages of this new weapon; but as the Turks have not been able to make use of it, battles among the clouds still remain a problem of the future. The rapid aeronautical preparations now making in Europe suggest that it will not long remain unsolved.

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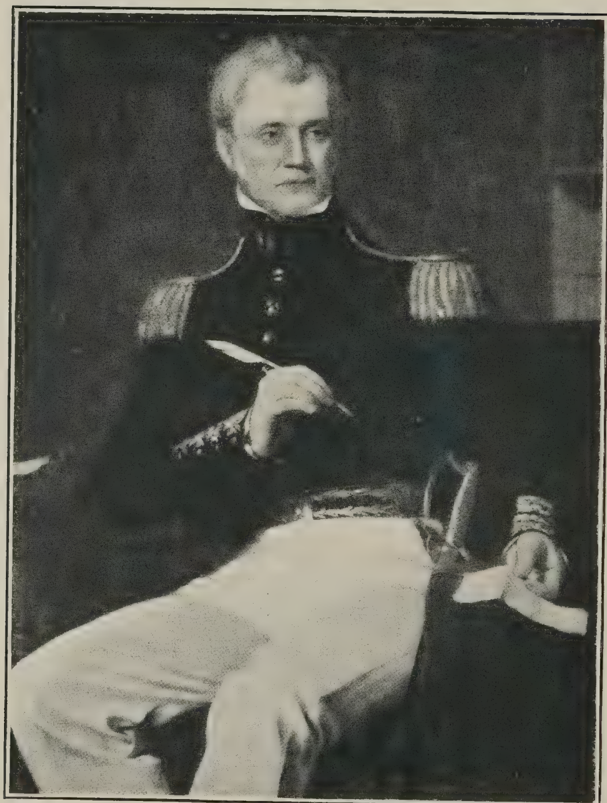

BRIG. GEN. SYLVANUS THAYER
The Father of the United States Military Academy
Commanding the Corps of Engineers
United States Army, 1857-1858

BORN 1785—DIED 1872

The Development of Regulation Works and the Use of Concrete in the Improvement of the Missouri River

BY

Maj. EDWARD H. SCHULZ*
Corps of Engineers

The improvement of the Missouri River, other than snagging, dates from 1878; snagging was carried on at an earlier date. In its natural state, the width of the river varies from 600 to 5,000 feet. The distance between bluffs is generally from 2 to 4 miles, but at Sioux City, 811 miles above the mouth, it is as great as 17 miles. The mean discharge at Kansas City, 392 miles above the mouth, is about 71,500 second-feet, while the low-water discharge during navigation season is about 27,000 second-feet. The extreme flood discharge of 1903 was estimated at Kansas City to be 750,000 second-feet. The slope of the river is about .88 foot per mile. The regulated channel, to produce a 6-foot depth at low water, having regard for flood provision, should be about 1,500 feet in width. The length of the continuously navigable portion from the mouth to Fort Benton is 2,285 miles.

The valley of the river is composed of alluvium, which is readily acted upon by the river currents. In its winding and wandering course between the bluffs, the river is continuously eroding and redepositing this soil, thus changing and forming bars and interfering with navigation. It has been estimated that sediment to the extent of about 400,000,000 tons annually is brought down by the Missouri and washed into the Mississippi River.

To improve and regulate the river, snagging, bank protection, and channel contraction have been used. Dredging has practically never been utilized on the Missouri River, though its use may be necessary to overcome temporary obstructions to navigation. Snagging has been carried on for many years and has been of great

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*In charge of the improvement of the Missouri River and its tributaries.

value in opening channels and giving greater safety to navigation.

Two principal methods of bank protection and regulation have been developed—the revetment to protect and hold existing banks and the longitudinal, diagonal, and spur dikes to hold banks, guide the channel and contract the flow. These are the result of nearly thirty-five years of experiment and experience under all sorts of conditions and requirements.

The following account, with illustrations, Figs. 1 to 9, showing the various steps leading to the present improvement, is taken from a report by a Board of Engineer Officers* dated November 10, 1910, H. R. Document No. 46, 62d Congress, 1st session.

Fig. 1. *Original Brownlow Weed.* A number of small trees or a lot of brush is tied to a rope and this entire bunch is fastened to a float anchored in the stream by means of sacks or a network filled with brick or stone. This device was used on the rivers in the East Indies by the Royal Engineers previous to 1875.

Fig. 2. *Adaptation of Brownlow Weed.* This was used at Nebraska City, Nebr. One rope was substituted for the two ropes of the original Brownlow weed. The anchor was attached directly to the upstream end and the float to the downstream end. This was done for economical reasons, as well as to prevent the ice from destroying the entire dike. Sunken weeds were found necessary to hold anchors. The dike was 758 feet long and caused considerable fill and head above it. Cost, $10 per linear foot.

Fig. 3. *Weed with Cottonwood Sapling Core.* At Council Bluffs, Iowa, in 1879, cottonwood saplings in one or more lengths were substituted for the rope core with good results. The system of anchoring was also changed. A rope of proper length was firmly anchored at one end of the line of the proposed dike. To this, at proper intervals, the weeds were successively attached and thrown overboard. Each weed had its own anchor, but the whole system was connected, thus reducing the total weight of anchorage required, while the weeds could be located with more certainty. This dike was 1,200 feet long and was regarded at the time as perfectly successful.

Fig. 4. *Stiff Weed.* The stiff weed consists of two rows of brush interlocked at right angles, so that two consecutive pieces

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*Maj. Chas. Keller, Maj. M. L. Walker, and Maj. E. H. Schulz, Corps of Engineers.

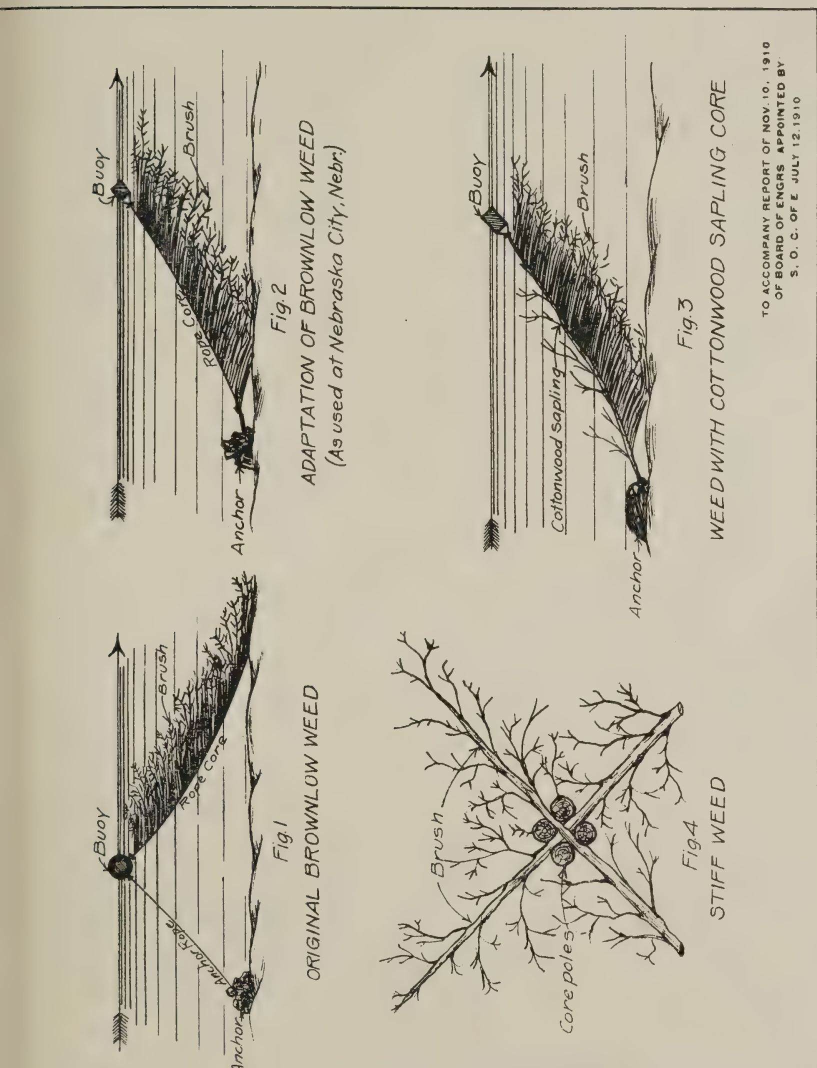


Fig. 1
ORIGINAL BROWNLOW WEED

Fig. 2
ADAPTATION OF BROWNLOW WEED
(As used at Nebraska City, Nebr.)

Fig. 4
STIFF WEED

Fig. 3
WEED WITH COTTONWOOD SAPLING CORE

of brush, one from each row, form the letter X. A slim pole of the required length of the weed (spliced, if necessary) is placed in each of the angles and the four poles are bound tightly together. The dike acted like an abattis and was used to close a chute near Fort Leavenworth, Kans., in 1879.

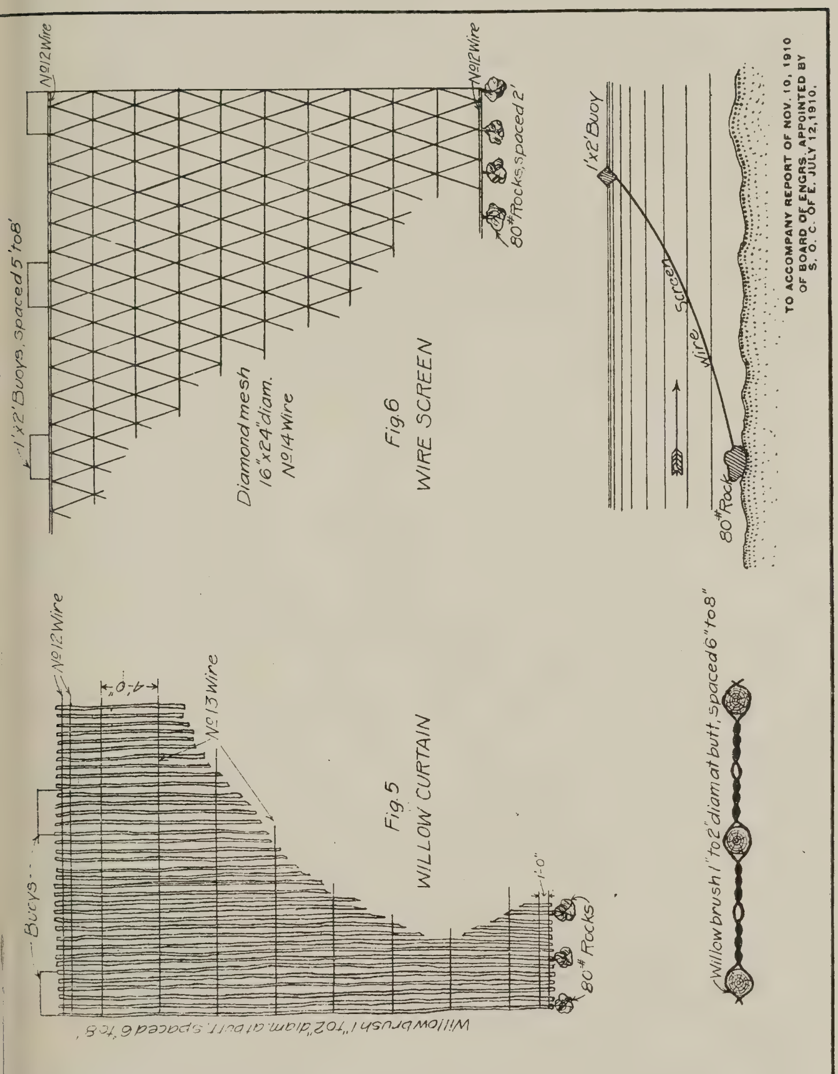
Fig. 5. *Willow Curtain with Buoy Supports.* The willow curtain dike is a grating or curtain made of brush held in place by anchors at the upstream end and supported by buoys at the other end. The curtain consists of a number of pieces of brush, 1 to 2 inches in diameter at the butt, and parallel to each other, about 6 to 8 inches apart, bound together by wires which were placed under and over the brush every 4 feet and were twisted together between the poles. This form was used in the construction of a dike at Nebraska City, Nebr., in the summer of 1879.

Fig. 6. *Wire Curtain with Buoy Supports.* The dike is essentially a wire screen of coarse mesh held in place by weighting one edge with rock and supporting the other edge with buoys. First introduced at Wyoming, near Nebraska City, Nebr., in the summer of 1879.

Fig. 7. *Screen Dike with Supports of A-shaped Pile Bents.* Instead of buoy supports for the curtains, piles were driven at an inclination and in pairs like the letter A. At first a log waling was placed from bent to bent, to which the curtain was attached. Later, wire ropes were used. Bents were spaced 25 to 30 feet apart, and a pile added about every 200 feet, forming a tripod. This was introduced at Nebraska City in 1879, and was used at St. Charles, Mo., in 1881.

Fig. 8. *Screen Dike with Tripod Support.* This dike consists of a wire screen anchored on the bottom and supported on a wire rope suspended from pole tripods. The screen was similar to those used in the floating dikes. The tripods were made of hardwood poles about 6 inches diameter at butts, securely wired to base of poles, forming an equilateral triangle. The tripods were anchored by attaching nets, holding nearly a yard of rock, to each leg of the tripod. This dike was built in the spring of 1881 at Cedar City, Mo.

Fig. 9. *Bar Dike.* This dike differs from the one shown in Fig. 8 only in its support and in the method of construction. A number of the supports shown in the sketch were set on a bar about 20 feet apart, over which a cable was stretched to hang the wire net. The net was 25 feet wide, similar to the screens pre-


Nº12 Wire
1'x2' Buoys, spaced 5' to 8'
Nº12 Wire
80# Rocks, spaced 2'
Diamond mesh
16"x24" diam.
Nº14 Wire
Fig. 6
WIRE SCREEN
1'x2' Buoy
Wire Screen
80# Rock
TO ACCOMPANY REPORT OF NOV. 10, 1910
OF BOARD OF ENGRS. APPOINTED BY
S. O. C. OF E. JULY 12, 1910.
Nº12 Wire
4'-0"
Nº13 Wire
Buoys
Fig. 5
WILLOW CURTAIN
1'-0"
80# Rocks
Willow brush 1" to 2" diam. at butt, spaced 6" to 8'
Willow brush 1" to 2" diam at butt, spaced 6" to 8"

viously described, and was anchored with rocks at the lower edge. It was used near Nebraska City, Nebr., in 1881.

Fig. 10. *Pile Dike Screened with Wire Netting.* Dike consists of two rows of vertical piles 14 feet apart in each direction, with longitudinal wale and diagonal tracing, screened with wire netting of diamond-shaped mesh (12 by 12 inches). The netting was fastened to the piles and waling by wire and staples, and was not anchored. Used at Sioux City, Iowa, in 1882.

Fig. 11. *Pile and Mattress Dike.* This dike had two rows of piles, 10½ feet apart, braced with walings 4 by 8 inches, and diagonal and transverse braces 4 by 6 inches, bolted and lashed to the heads of the piles. A foot mat, 20 feet wide and 16 inches thick, was placed on the bottom of the river to prevent scour. In front of the double row of piles on the upstream side a continuous woven mattress was constructed, 30 to 35 feet wide. This was used at the East Bottoms, Kansas City, Mo., in the spring of 1887.

Fig. 12. *Pile Dike with Woven Mat Screen.* This dike consists of two or three rows of piles with two wales of cottonwood, cross and diagonal braces, securely bolted and lashed, protected from scour by a foot mat and screened with a woven mat. The foot mats are continuous throughout the length of the dike, and consist of bundles of willow brush held between two horizontal frames made of yellow pine. This was used at Little Platte Bend, near Kansas City, Mo., in the spring of 1889.

Fig. 13. *Standard 3-row Timber Pile Dike.* The present standard dike consists of three rows of piles, 10 feet center to center in each direction, having two horizontal systems of bracing, a foot mat and a screen of curtain poles. The outer (or river) row of piles has a penetration of 25 feet and the other two rows 20 feet. Waling timber, 6 by 8 inches, connects the outer row of piles, and 4 by 8 inch timbers at each bent. All the other timbers are securely fastened to the piles with ¾-inch bolts. The foot mat is of woven willow construction, 75 feet wide, reinforced with double ⅜-inch strand, both longitudinally and crosswise. The screening poles are 2 to 4 inches in diameter at butts and are spiked to the waling timbers on the middle row of piles, 12 inches apart.

Fig. 14. *Standard 3-row Concrete Pile Dike.* The general plan of this dike is the same as the standard 3-row wood pile dike, the only difference being the substitution of reinforced concrete piles and bracing for the timber. The piles are 12 inches square, re-

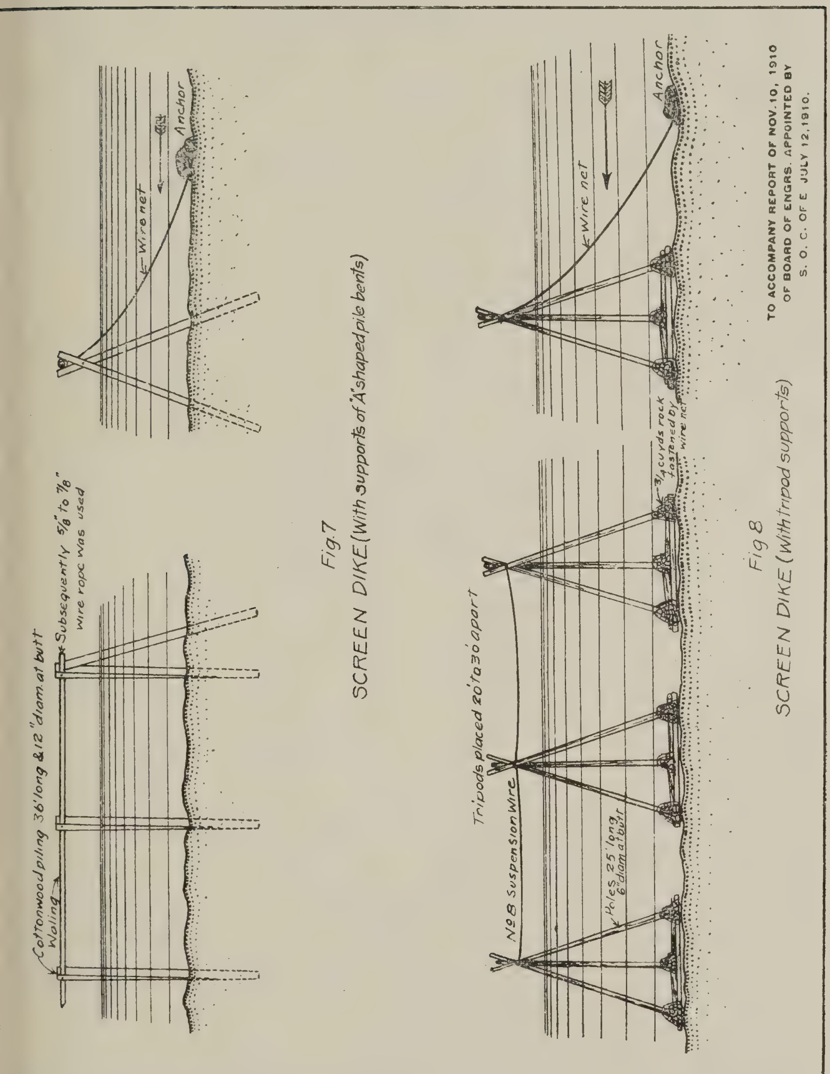


Fig. 7
SCREEN DIKE (With supports of 'A' shaped pile bents)

Fig. 8
SCREEN DIKE (With tripod supports)

inforced with four $\frac{3}{4}$-inch bars, and are cast in forms at the site of the dike. The braces have a section 8 by 8 inches, are reinforced with four $\frac{3}{8}$-inch bars, constructed in place flush with the face of the piles. The first dike of concrete piles was constructed at St. Joseph, Mo., in the fall of 1908. A dike similar to above, but of 2-row work, 800 feet long, was completed at Fort Riley, Kans., in 1910.

Fig. 15. *Standard Woven Willow Mattress.* This is the present standard bank protection. The mattress is of continuous woven willow construction, 86 feet wide, the upper edge being placed a few feet above standard low water. The willows are 10 to 25 feet long and woven with a diamond-shaped mesh with selvage edge. The mattress is weighted with rock and sunk, and the bank above low water is graded to a slope of 1 on 3 and paved up to high water. This is the final product of years of experimentation in bank protection. The first form was a grillage of brush laid as sectional mattress; then a woven willow mattress was used at Vermillion, S. D., in 1879. The expedient of fastening the brush to slopes by wire and stakes was also considered. It was, however, thought that the willow construction was unnecessarily heavy and a lighter one of wire was evolved, on which a suspended wire fence of variously formed meshes was placed in front of caving banks, and for a while had apparent success in protecting the bank, stopping erosion and filling pockets. But this type of protection was soon destroyed. The continuous woven mattress first used at Vermillion then came into use again. Later, the mattress was omitted on the slope above low water and paving substituted. The mattress was also strengthened by longitudinal and transverse wire strands, thus leading up to the present standard type.

The standard timber pile-dikes built along the Missouri River have an average life of from seven to ten years. Usually the cause of their removal in that time is decay about the water line, but very often floods, erosion, floating ice and drift destroy them before they actually wear out. With the idea that concrete would resist both deteriorating influences of rot and exterior abrasive or destructive action, the Chief of Engineers authorized the construction of the concrete dike, in order to test the value of concrete, in Elwood Bend above St. Joseph, Mo. This was built in 1908, and is believed to be the first concrete dike of its kind in any river improvement.

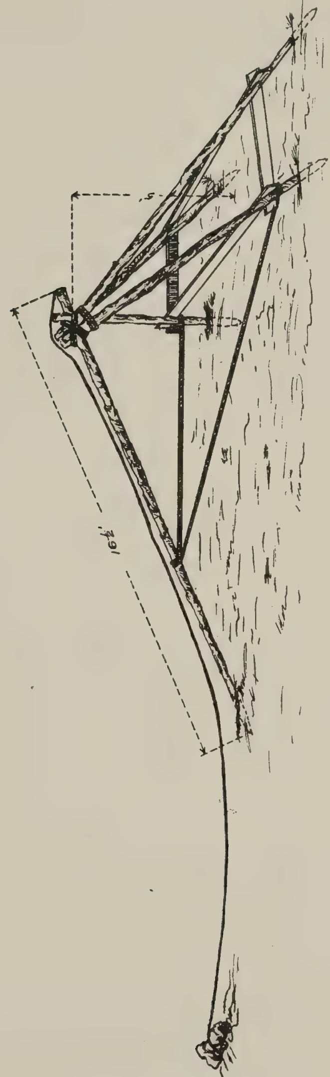


Fig. 9
BAR DIKE SUPPORT

TO ACCOMPANY REPORT OF NOV. 10, 1910
OF BOARD OF ENGRS. APPOINTED BY
S. O. C. OF E. JULY 12, 1910.

The proposals called for supervising the work of manufacturing the piles and for furnishing forms and reinforcing materials, the United States to furnish other material and make and drive the piles. All piles were to be cast on the ground and were to be driven by jet and hammer combined. Under this specification bids as low as 80 cents per linear foot were received. Various methods of casting and driving were considered, but were discarded for reasons connected with the peculiar conditions involved. The number of piles used was so small that any method requiring much apparatus would be prohibitive as to cost.

The following gives the more important details of the dike:

Total length of dike 150 feet, of which 40 feet was timber nearest the shore, and 110 feet of concrete piles.

Total piles driven, 36. Total linear feet, 1,457.

Length of piles, 32 to 50 feet.

Penetration, average, 21 feet.

Elevation of top of piles, 10 feet above standard low water.

Cross section of piles: 14 inches, square on top, 8 inches square at base.

Reinforcement: Four bars of 1-inch square steel, ½ inch round tie steel, 18 inches center to center.

Forms, 2-inch yellow pine.

Mixture: 1 part cement, 2 parts Missouri River sand, 4 parts crushed rock, largest size 1 inch.

Weight of 50-foot piles, 8,700 pounds.

Age, set ten days before driving.

The piles were cast on shore about 6 feet above a barge, on which they were skidded preparatory to being driven. On account of the weight of the pile, a wire cable, with a single and double block, was used in the handling, slinging each pile at the large end and at about 15 feet from the small end to take care of the deflection under its own weight. It was found that a pile could bow 5 inches in the 50 feet without injuring it.

In driving in sand only, the piles were jetted down in about five minutes, by means of a jet attached to a 1¼-inch pipe fastened along the pile. But in that part near the shore covered with rock and débris, considerable difficulty was experienced and the hammer had to be used. At first it was intended to build the piles only of concrete, tying them together with timber, but in a portion of the

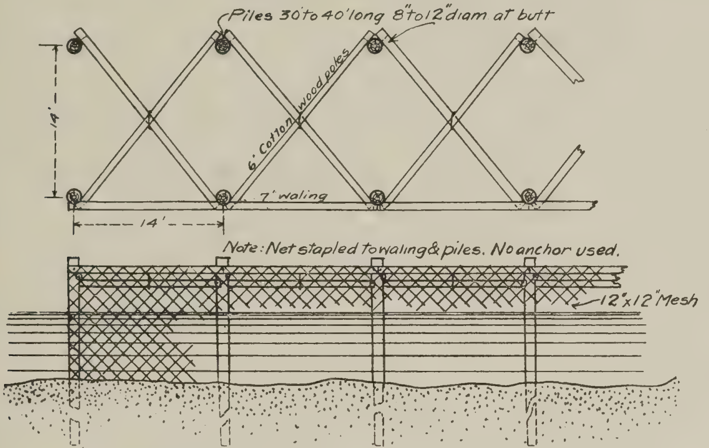


Fig. 10
2-ROW PILE DIKE (With wire screen)

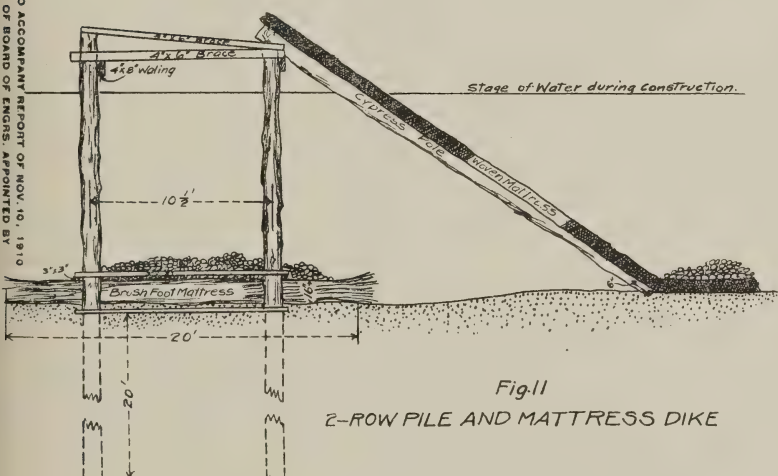


Fig. 11
2-ROW PILE AND MATTRESS DIKE

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dike concrete cross beams were substituted for the timber, as will be noted in Fig. 17.

The actual cost to the United States was $1.36 per linear foot of pile, which, after deducting the expense of special supervision, would make the cost about $1.00 per linear foot, a figure which could be expected to be reached by the Government in future work. The itemized cost of the construction is as follows:

Invoice for supervising, forms and steel rods	$1,200.00
Portland cement, 86¾ barrels, at $1.25 barrel	108.44
Crushed rock, 111,800 pounds, at $1.30 ton	72.67
Sand, 32 cubic yards, at $0.20 per cubic yard	6.40
Labor, making forms	117.00
Labor, making piles	257.70
Labor, driving piles	215.00
Total	$1,977.21

This dike has stood the test of floods, ice gorges, and débris for the last three seasons and has suffered no loss, except in parts of the wood bracing destroyed by the ice. Photographic views are herewith in Figs. 16, 17, and 18. Fig. 16 shows work during construction, Fig. 17 the completed dike, and Fig. 18 the dike after an attack of ice and débris.

The next construction was a concrete dike closing off the left hand chute of the Republican River above the Washington Street bridge at Fort Riley, Kansas. The map, Fig. 19, shows the general conditions. (See, also, illustrations, Figs. 20 to 24.) The river above the bridge had considerably eroded the left bank, and continued to such an extent that not only was the bridge threatened, but also a portion of the United States Military Reservation, amounting to about 20 acres. The normal width of the Republican River is from 400 to 600 feet. The shore length to be protected was about 3,600 feet, and to revet this entire line would have cost approximately $25,000. Such work would hold the present left bank, but would not materially assist in regulating the river or placing it in its old position.

It was therefore considered advisable to construct two dikes, one closing the left chute, 800 feet long, extending from the main bank to head of island. This would not only close the chute, but cause the sand bar behind the island to fill in. The other dike, 400 feet long, was to extend from foot of island toward the north end of

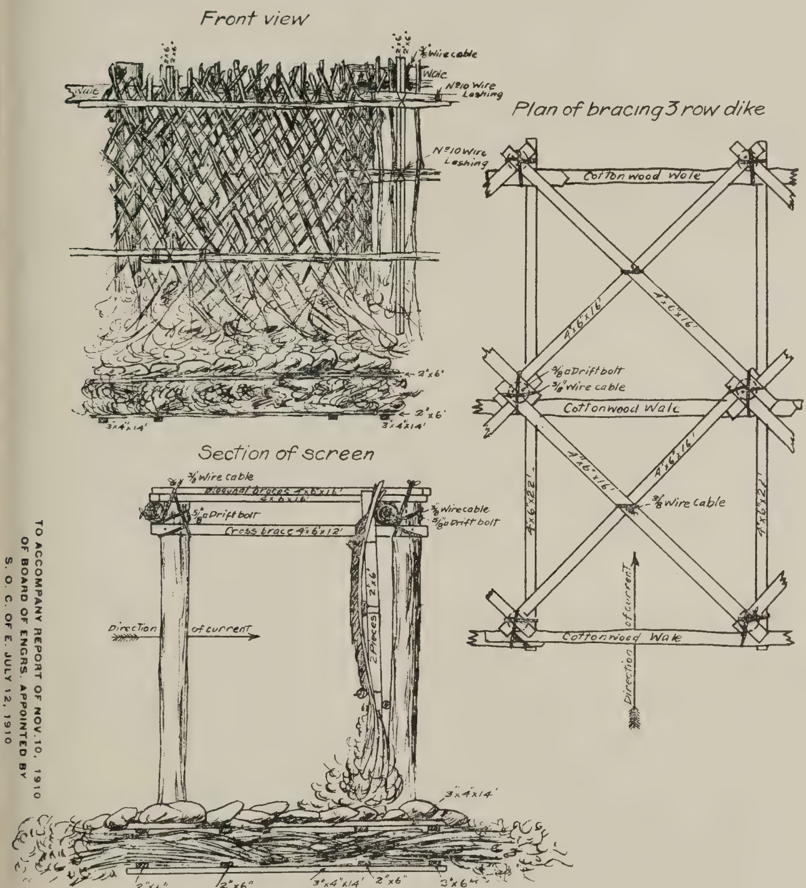

Fig. 12
DIKES WITH WOVEN SCREEN

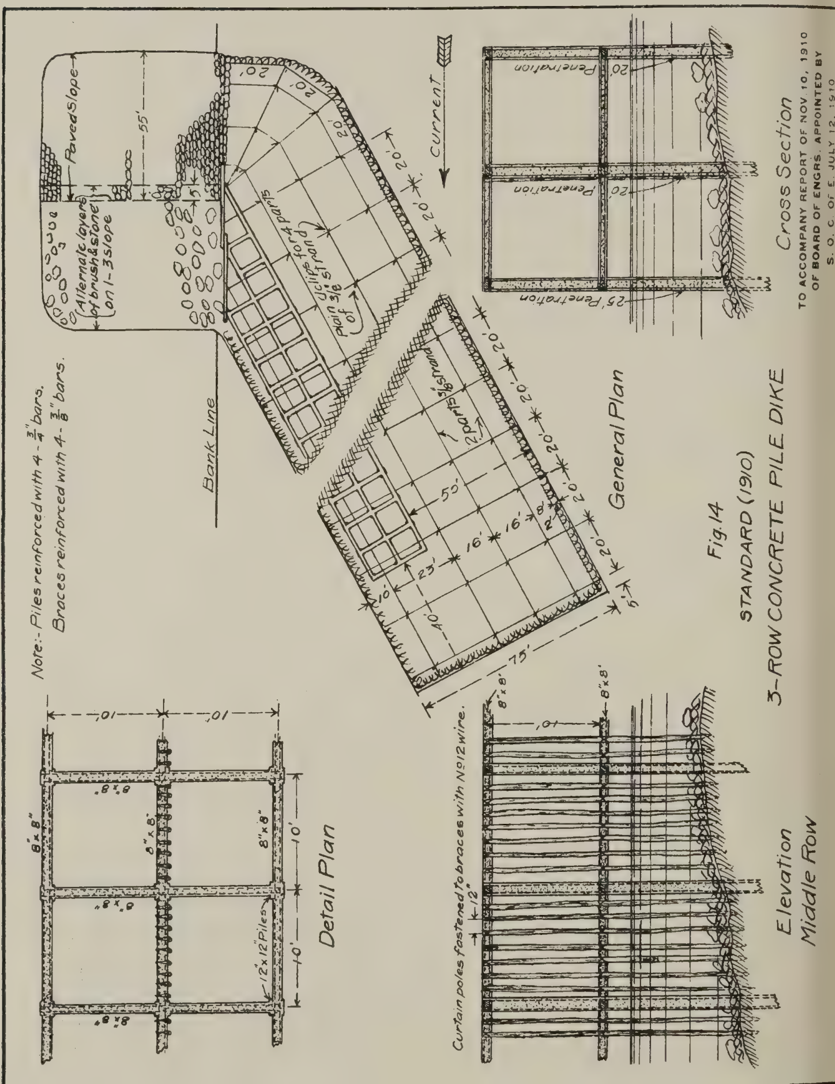

Fig. 14
STANDARD (1910)
3-ROW CONCRETE PILE DIKE

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freight charges and long haul by team and the erection of scaffolds for weaving across the water. The screening poles were wired into place on the rear concrete bracing.

The rise of June, 1910, caused no injury to either dike, but damaged the connections with bank of the upper dike as follows:

At the upper end a 40-foot break occurred in brush and rock fill, which was readily repaired. At lower end a 160-foot opening between the dike and head of island was eroded. The dike itself was not injured. To close this opening and to extend the dike into island beyond further erosion, an additional construction of 300 linear feet was made in the fall of 1911. A 50-foot mattress was woven ahead of the pile driving across the gap. That for the land dike was woven in place after pile driving and was 40 feet wide.

In addition to the willow foot mattress, a system of flexible concrete blocks was laid for 23 feet at the end of the dike. The blocks are 18 inches square and 4 inches thick, reinforced by four crossed galvanized-iron bars of $\frac{1}{4}$-inch diameter, with an eye turned on each end to connect adjacent blocks. The eyes are 19-inch centers, and the connection between blocks is made with $\frac{1}{4}$ by 1 inch galvanized machine bolt. A total of 300 blocks were cast at a cost of \$0.515 per block laid in place. The cost of the original two dikes, 800 and 400 feet, was approximately \$11.37 per linear foot. The cost of the 300-foot extension (being a smaller job), together with repairs as noted, was approximately \$21.80 per linear foot. The average cost per linear foot of entire 1,500 feet of 2-row concrete dike, with repairs, etc., was approximately \$13.03 per linear foot, comparing favorably in cost with the usual timber pile dike and exceeding it in durability.

In addition to the dike construction, 300 feet of standard willow revetment was placed above upper end of 800-foot dike, and 100 feet below the lower end at head of island. The mattress was 45 feet wide and the cost \$6.40 per linear foot.

The entire system has gone through the present winter of ice, débris, and flood without any injury. The fill in rear of island below dike A is now within 6 to 8 feet of top of dike, and it requires a considerable rise to go through the chute.

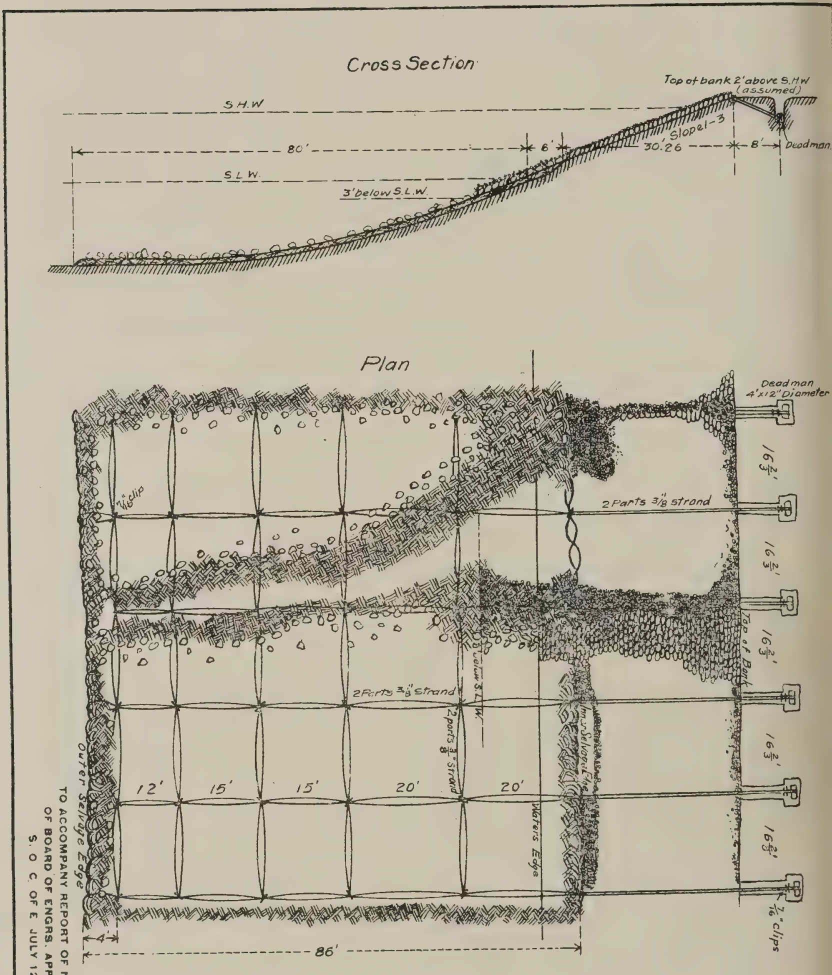


Fig. 15

STANDARD REVETMENT

Five illustrations of the Fort Riley work are herewith, as follows:

Fig. 20. Pile casting yard.
Fig. 21. Downstream view of upper dike from land side.
Fig. 22. Downstream view of lower dike from river side.
Fig. 23. View of flexible concrete block mattress.
Fig. 24. View of revetment, lower end of upper dike.

The third concrete dike construction was on the left bank of the

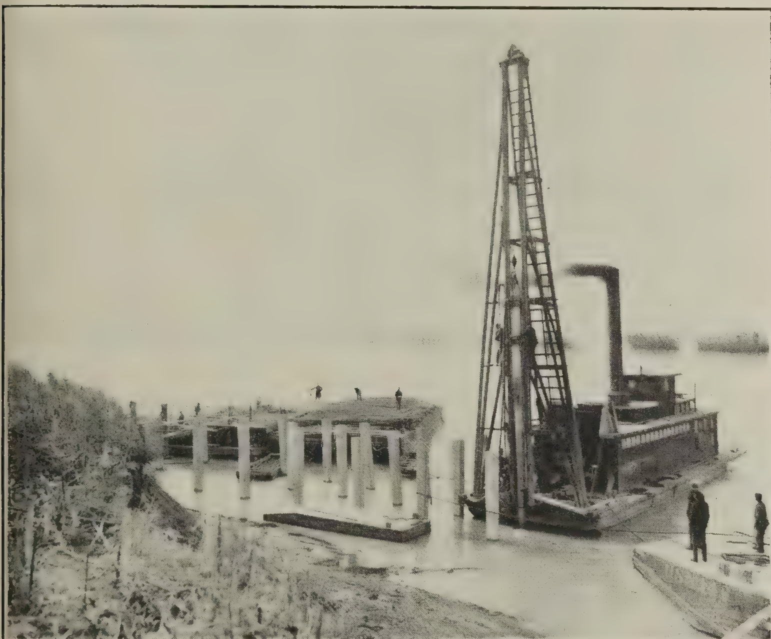


Fig. 16. Driving concrete piles at Elwood Bend, Mo.

Missouri River above St. Joseph, Mo. This was built during the season of 1910 to protect an eroded pocket in an original revetment. The dike is 500 feet long and consists of three rows of concrete piles, spaced 10 feet center to center, braced with reinforced concrete beams, and having the usual foot mattress, pole screen, paving, etc., of the wooden dike. The piles are 12 by 12 inches, reinforced with 1-inch twisted bars tied with ¼-inch round iron every 24 inches, and are from 35 to 50 feet long. The penetration is from 20 to 28 feet, and the tops are about 15 feet above standard low water. The braces are 8 inches wide, 9 inches deep, and ex-

Fig. 17. Finished dike at Elwood Bend, looking downstream.

tend from pile to pile (about 9 feet), and are reinforced with four ¾-inch twisted bars tied with ¼-inch round iron every 24 inches.

The willow screening poles were wired to the braces of the center row of piles with No. 12 wire. In casting the piles, holes were left through which the reinforcing bars of the lower braces were placed. The reinforcing bars in the piles were allowed to protrude about 6 inches at the top and the upper bracing system was built over the tops of the piles, thus bracing the entire structure securely together. One hundred and fifty-three piles, 6,564 feet in

Fig. 18. Dike at Elwood after a siege of flood, ice, and drift.

all, were cast and mainly sunk by jet. A view of this dike after construction is shown in Fig. 25.

Summary of Cost.

Casting piles, 6,564 linear feet, at $0.921 per foot	$6,048.20
Sinking piles, 6,396 linear feet, at $0.545 per foot	3,489.40
Bracing dike, 4,634 linear feet, at $0.85 per foot	3,729.60
Weavin g and ballasting mattress, 510 linear feet, at $5.85 per linear foot	2,984.10
Screening dike, 500 linear feet, at $0.43 per foot	215.40
Filling roots, 2 at $59.57 each	119.15
Total	$16,585.85

500 linear feet 3-row concrete dike at $33.17 per linear foot.

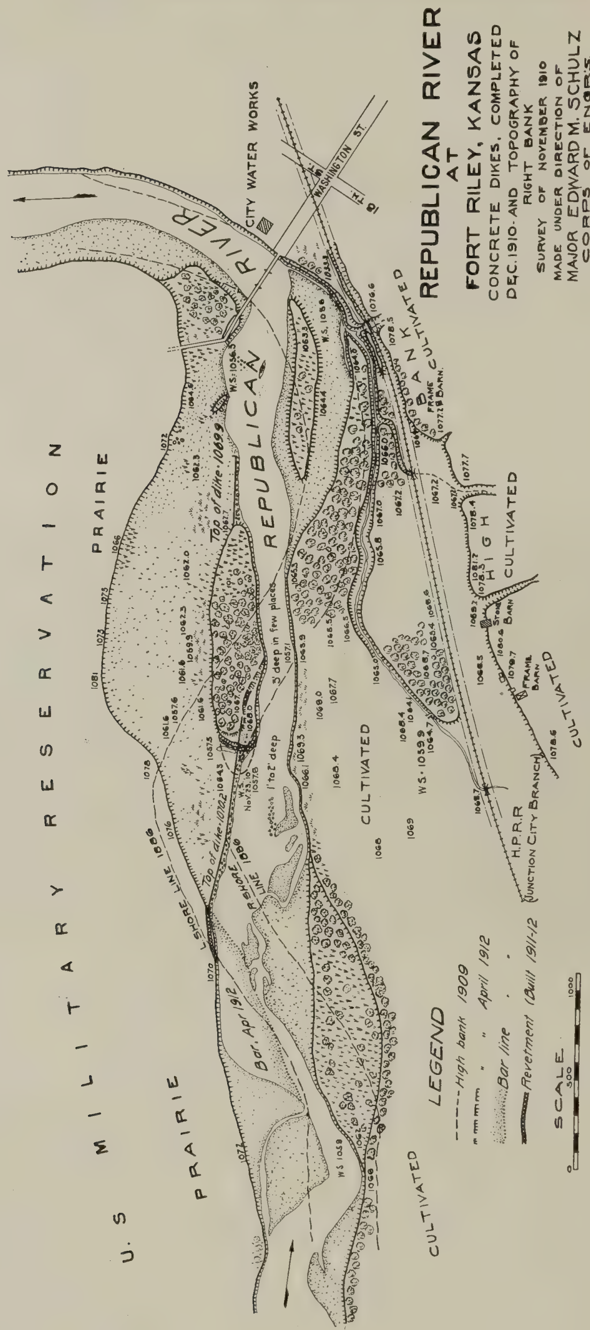

REPUBLICAN RIVER
AT
FORT RILEY, KANSAS
CONCRETE DIKES, COMPLETED
DEC. 1910. AND TOPOGRAPHY OF
RIGHT BANK
SURVEY OF NOVEMBER 1910
MADE UNDER DIRECTION OF
MAJOR EDWARD M. SCHULZ
CORPS OF ENGR'S.
U. S MILITARY RESERVATION
PRAIRIE
PRAIRIE
REPUBLICAN RIVER
CITY WATER WORKS
WASHINGTON ST.
CULTIVATED
CULTIVATED
CULTIVATED
CULTIVATED
CULTIVATED
HIGH BANK
Top of dike 1069.9
Top of dike 1070.2
L SHORE LINE 1886
R SHORE LINE 1886
Bar, Apr 1912
3' deep in few places
1' to 2' deep
FRAME BARN
STONE BARN
FRAME BARN
H.P.R.R
JUNCTION CITY BRANCH
LEGEND
High bank 1909
" " April 1912
Bar line
Revetment (Built 1911-12
SCALE
0
500
1000

Fig. 19.

This dike went through the floods and ice of 1910 and 1911 without mishap, but was injured in the ice gorge and flood of the spring of 1912, as shown in Fig. 26. The damage consisted in breaking off six piles and four braces at lower end, disconnecting two piles on outer row and eroding the brush and rock fill at lower end. The ice gorge broke at this locality and the crushing and pushing force was irresistible. The damage done on four wooden dikes a few hundred feet below was very much greater. The concrete dike

Fig. 20. Concrete pile yard; Republican River, Fort Riley, Kans.

will be repaired at once. The injury was a reasonable one and does not disprove the value of concrete for such structures.

A further authorization is the construction of a concrete dike at Commerce Point, Pelican Bend, on the right bank of the Missouri River near the mouth. The dike is to be 3-row, 4,500 feet long. The estimated cost is $25 per linear foot, or about $112,500.00 for the total.

Pelican Bend has always been a bad stretch. The fall of the river along the chute to be closed is 1.18 feet per mile, while

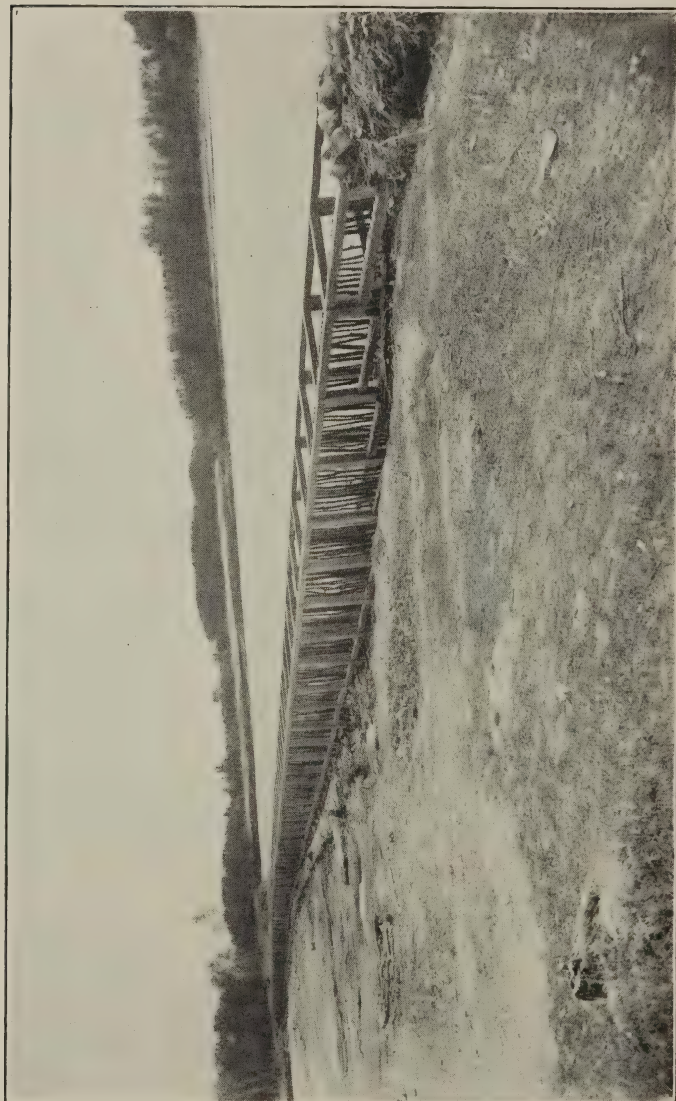
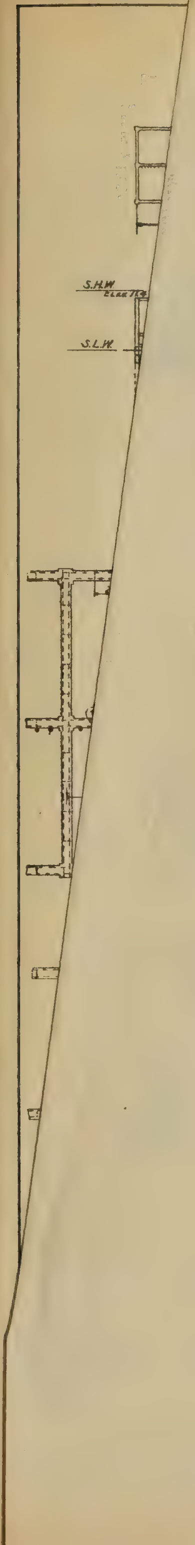
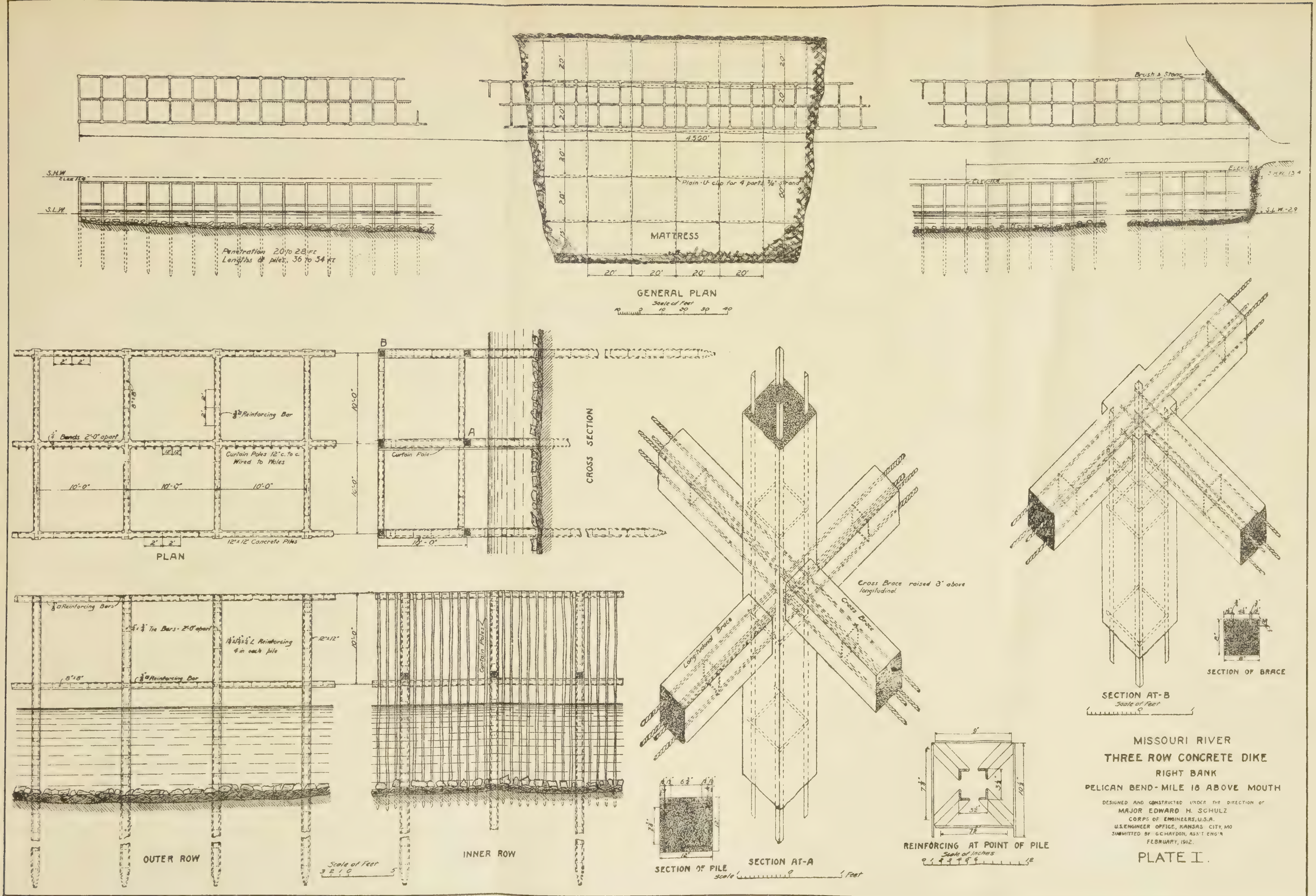


Fig. 21. Upper concrete dike, Fort Riley, Kans.; 1,100 feet. Looking downstream from land side.

WAR DEPARTMENT





MISSOURI RIVER

THREE ROW CONCRETE DIKE

RIGHT BANK

PELICAN BEND - MILE 18 ABOVE MOUTH

DESIGNED AND CONSTRUCTED UNDER THE DIRECTION OF
MAJOR EDWARD H. SCHULZ
CORPS OF ENGINEERS, U.S.A.
U.S. ENGINEER OFFICE, KANSAS CITY, MO.
SUBMITTED BY G. C. HAYDON, ASS'T ENG'R
FEBRUARY, 1912.

PLATE I.

Fig. 22. Lower concrete dike, Fort Riley, Kans. Looking downstream from river side.

around the bend it is 0.77 foot, which agrees nearly with the average river slope.

The dike is primarily for the rectification of the right bank, as it will close the main one of the numerous chutes in Pelican Bend. Its first service will be in closing the chute and the second will be to act as a permanent structure and serve as revetment, hence the cost of a wooden dike ($15.00 per linear foot) for closing purposes, and later the construction of revetment ($10.00 per linear foot) for permanency will be combined in the first cost of the concrete dike.

It is proposed to do this work by hired labor and Government plant. The concrete piles will be cast on the shore in the vicinity of the plant. The piles will be made 50 feet or longer and the mattress wider than usual in order to prevent all possible scour.

Plate I is a print showing details of construction.

CONCRETE BANK PROTECTION.

The standard revetment now employed consists of grading the bank to a 1 on 3 slope, laying a woven willow mattress, 86 feet wide, from about 3 feet above low water and extending out over the river bed, and ballasting and sinking same; also paving the graded bank from inshore edge of mattress to top of bank. The standard revetment has been the most durable revetment found. If, however, the paving (riprap) fails, or is injured by drift, the bank begins to erode and the willow mattress is lost and of no avail; and the injured portion generally requires repair and replacement.

To overcome this difficulty it was thought a substantial reinforced concrete revetment could be placed, extending from the top of the bank to near low water line and from this point a system of connected concrete flexible blocks about 24 inches square could be placed, extending out beyond the lowest water. This protects the top bank, and also the weak point of the willow mattress, which is alternately wet and dry.

There has recently been placed a similar system of solid concrete on the Kansas River for levee and slope revetment, together with flexible blocks at the bottom for under-water protection. This work was done by the Kaw Valley Drainage District and contemplates in all some 10 miles of revetment.

The first concrete revetment work on the Missouri River by the

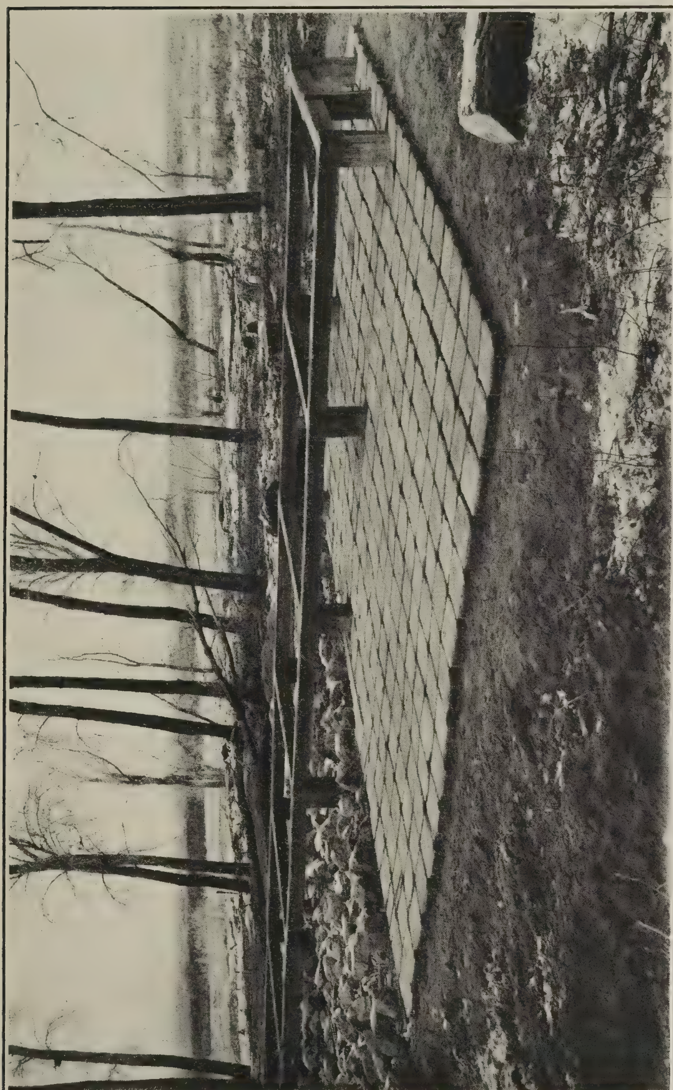


Fig. 23. Lower end upper concrete dike, Fort Riley, Kans.

United States was done at Gasconade, Mo. One thousand linear feet of bank was protected, as outlined above, as follows:

a. Standard Missouri River willow mattress 86 feet wide, extending from low water out over river bed, with ballasting. Actual length of mattress, 1,030 feet.

b. Reinforced concrete paving in 8-foot strips, laid from top of bank down to edge of mattress. Bank graded to 1 on 3 slope, average width of slope 49.5 feet. Concrete, 4 inches thick.

c. Flexible concrete block protection of reinforced concrete blocks, anchored together and to the solid concrete covering, extending from the bottom edge of concrete covering out on willow mattress, an average distance of 10 feet.

The concrete as actually placed at Gasconade was in 8-foot strips up and down the slope. That is, there were planes of separation up and down but not horizontally. The concrete, in addition to the strand of wire rope, had a reinforcement of fence wire covering the whole area and placed with 2-inch concrete above and below the wire. This reinforcement was a 4-inch triangular mesh of wire 1-16 to ⅛ inch diameter,laid the full width and length of each strip. Probably this reinforcement should be made stronger, and in future work the equivalent of ¼-inch wire will be used and the blocks separated each way about 6 to 8 feet.

The costs were as follows:

a. Willow mattress.		
757 cords brush, at $0.892	$675.87	
776 cubic yards ballast, at $0.746	579.09	
555 cubic yards spalling, at $0.702	389.70	
Wire strand and clips	204.43	
Towboat service	784.00	
Labor, superintendence and miscellaneous	669.59	
		$5,302.68
b. Concrete paving.		
1,156 cubic yards gravel, at $1.533	$1,772.35	
Grading bank, 1,000 linear feet, at $0.94607	946.07	
Reinforcement	749.32	
821¾ barrels Portland cement, at $1.15	945.01	
Labor, superintendence, etc.	1,881.35	
		$6,294.08
c. Flexible concrete blocks.		
120 cubic yards gravel, at $1.533	$183.96	
Reinforcement	221.65	
Portland cement, 182 barrels, at $1.15	209.30	
Labor, etc., making blocks	1,636.00	
Labor, etc., placing	685.12	
		$2,936.03
		$14,532.79

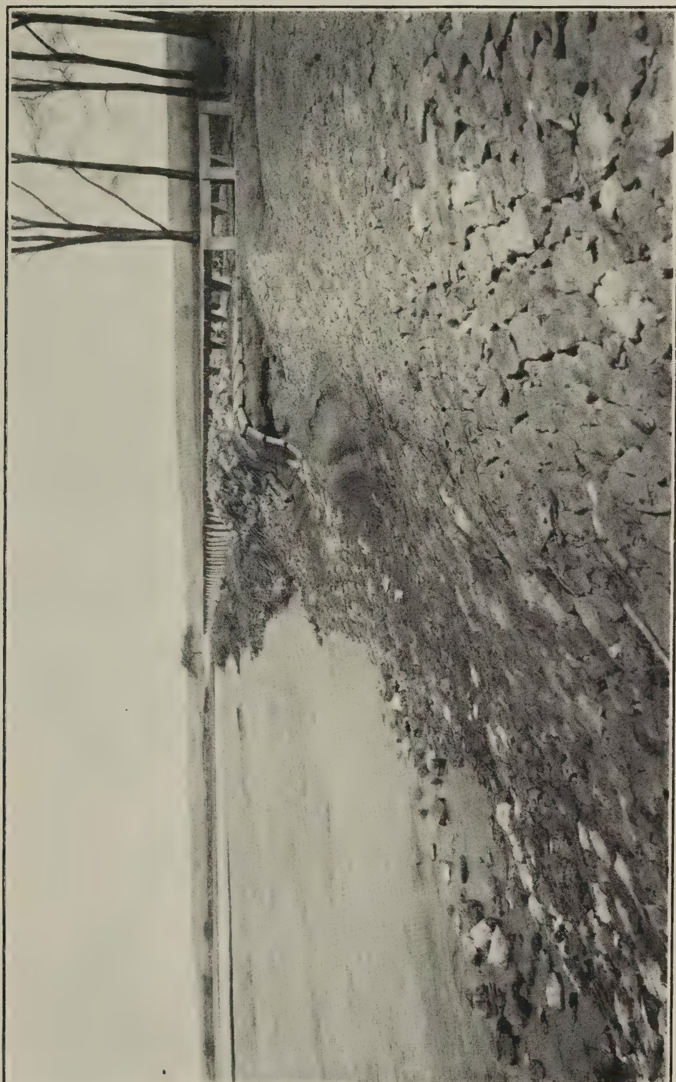


Fig. 24. Lower end upper concrete dike, Fort Riley, Kans., with extension of riprap revetment.

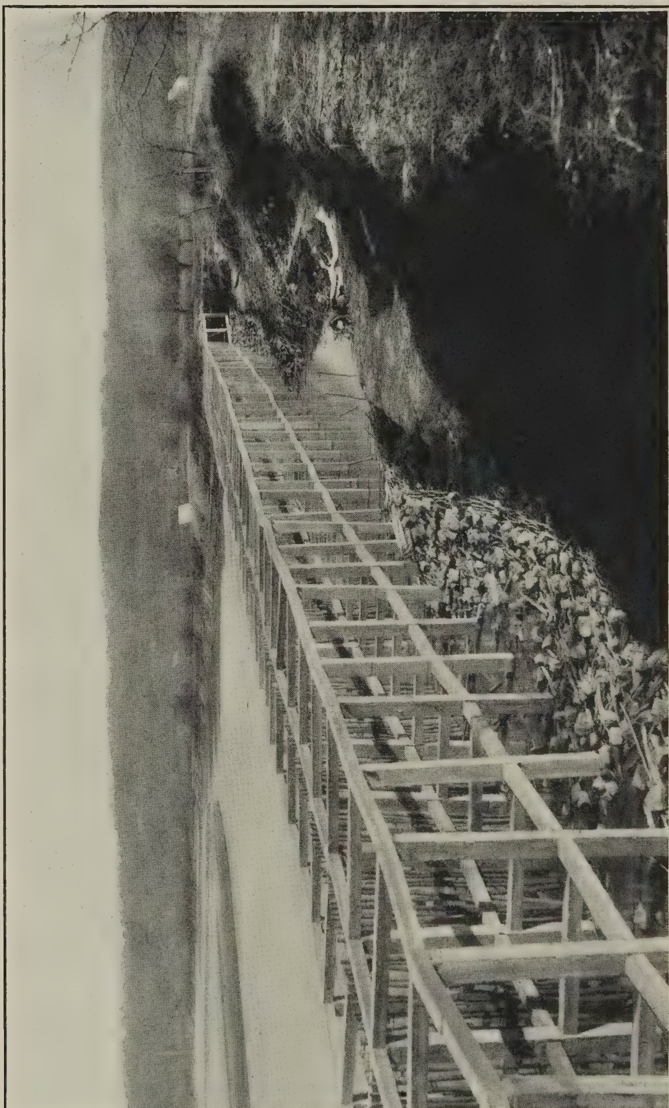


Fig. 25. Concrete dike, 500 feet, at Bonton Bend, St. Joseph, Mo.

Summary.

Weaving and ballasting mattress, 1,030 linear feet, at $5.147 per linear foot	$5,302.68
Concrete paving and grading, 1,000 linear feet, at $6.294 per linear foot	6,294.08
Flexible concrete blocks in place	2,936.03
	$14,532.79

The sum of $1,470.00 of the above was for extra slope fill and not properly chargeable to regular work. This leaves a new cost of $12,062.79, or about $12.00 per linear foot.

The usual cost of standard Missouri River revetment with 86-

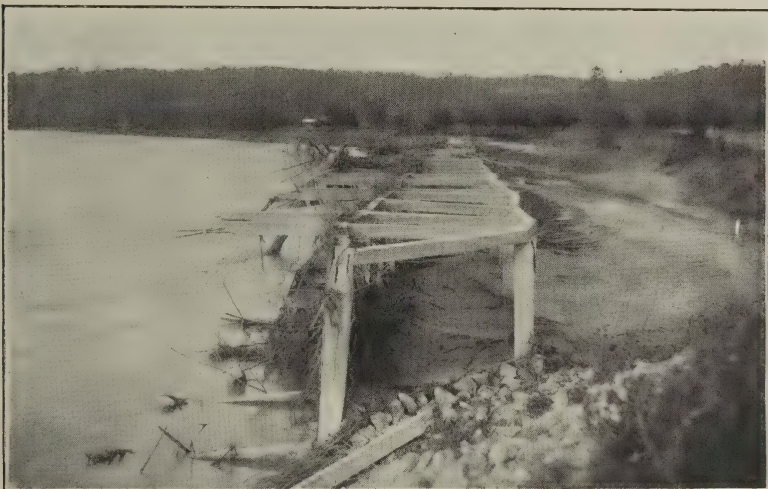


Fig. 26. Injury to concrete dike, Bonton Bend; flood and ice gorge of spring of 1912.

foot mattress, and bank graded to 1 on 3 and paved with riprap stone is from $8.00 to $10.00 per linear foot. Revetment work is usually done on a linear foot measurement, the number of square yards of bank paving depending on the height of bank above low water. The bank at Gasconade was of the usual height if not somewhat greater. The average area per linear foot of bank for a 1 on 3 slope and 86-foot mattress is 14 to 16 square yards, and its cost is 50 cents to 62½ cents per square yard.

The revetment at Gasconade has been completed and presents a most satisfactory appearance, and is believed to warrant its use on a larger scale. Figs. 27 and 28 show views of the completed work.

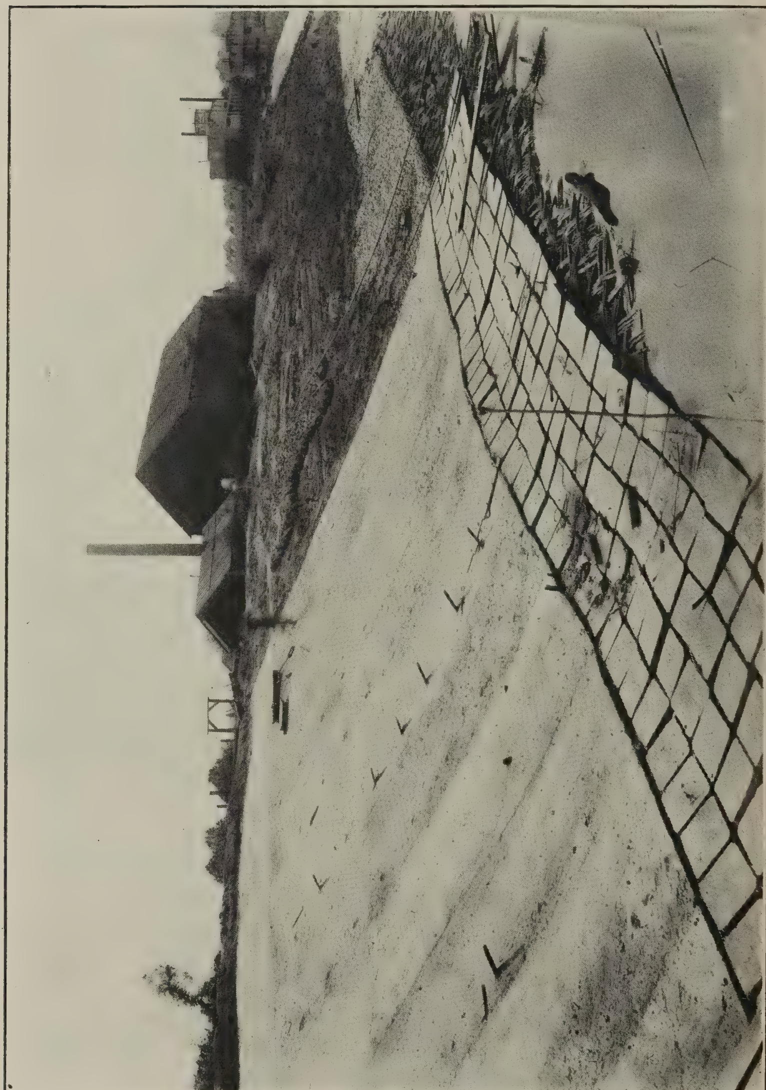
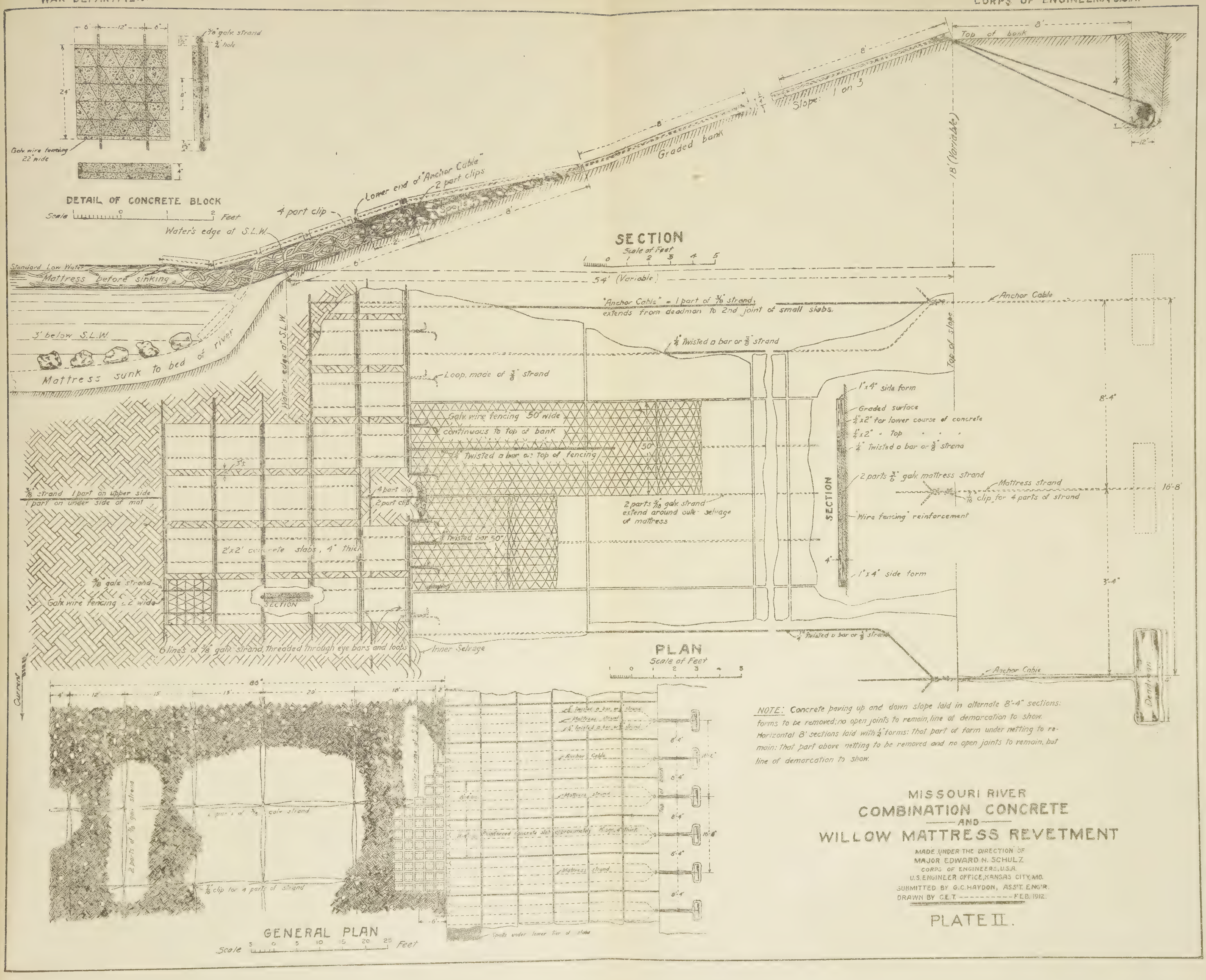


Fig. 27. Upper end concrete revetment at Gasconade, Mo.



NOTE: Concrete paving up and down slope laid in alternate 8'-4" sections: forms to be removed; no open joints to remain, line of demarcation to show. Horizontal 8' sections laid with 1/4" forms: that part of form under netting to remain: that part above netting to be removed and no open joints to remain, but line of demarcation to show.

MISSOURI RIVER
COMBINATION CONCRETE
AND
WILLOW MATTRESS REVETMENT

MADE UNDER THE DIRECTION OF
MAJOR EDWARD H. SCHULZ
CORPS OF ENGINEERS, U.S.A.
U.S. ENGINEER OFFICE, KANSAS CITY, MO.
SUBMITTED BY G.C. HAYDON, ASS'T. ENG'R.
DRAWN BY C.E.T. ---------- FEB. 1912.

PLATE II.

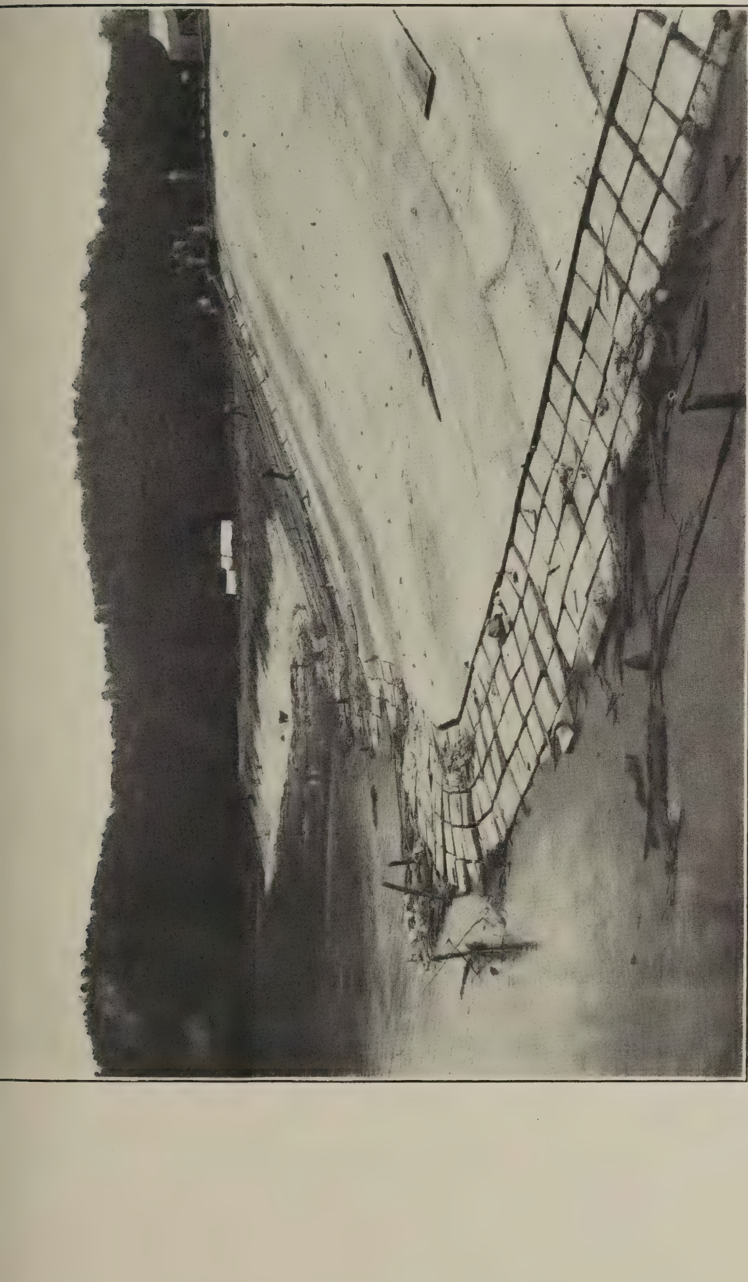


Fig. 28. Concrete revetment at Gasconade, Mo. Looking downstream.

The War Department has recently authorized the construction of 12,000 feet of combined willow mattress and concrete revetment on the right bank of the Missouri River near Berger, Mo. It is believed that this work will be on a large enough scale to reduce the cost to a minimum and at the same time will be a fair test of its value compared with riprap protection. The details are the result of the previous experience at Gasconade, and are embodied in Plate II herewith.

The assistant engineers on the Missouri River work are Mr. Geo. C. Haydon, Kansas City to the mouth, and Mr. F. W. Honens, Kansas City to Fort Benton.

Authorization of Public Works in France

BY

Maj. F. A. MAHAN,
Corps of Engineers, Retired,
of France

Applications for the improvement of a river or a port may emanate in any of the following ways:

1. The parties interested in the improvement may send their request directly to the Government, which, in such cases, is represented by the Minister of Public Works.

2. The parties interested may act through their chosen representatives.

3. The engineers in charge of the services have the right to present to the higher authorities any defects or insufficiencies of their works.

Whichever of these ways may be adopted, the Administration of the Ponts et Chaussées may call on the engineers to prepare a preliminary project showing the work to be done and giving an approximate estimate of the cost, or the engineers may, of their own initiative, present such a project.

The admirable maps of France, prepared by the General Staff of the Army, and the great network of levels (Nivellement général) which covers the entire country, are quite sufficient for the preparation of such a project, consequently no special fieldwork is required and the preliminary plans and estimates can be perfectly well prepared in the engineer's office.

This preliminary project is sent to the Central Administration which, if it finds that the project has been well prepared, may order it to be *taken into consideration.*

If the project be not very important and, especially, if there be no private property to be acquired, the Minister of Public Works may direct that the project be carried out at once or that it be postponed for a longer or shorter time, depending upon the urgency of the works and upon the funds which are available. Under

any circumstances, a *final* project must be prepared by the engineers before any work can be undertaken. This final project must be so complete that it can be used for obtaining bids or for making an informal agreement for carrying out the work. The work may also be executed by day's labor, the Government paying directly from its own funds.[1]

If the project involve any large cost (the limit for this being placed at 50,000 francs) or, if land has to be acquired, no matter how great or small the cost of the work may be, recourse must be had to a presidential decree, which is issued after a consultation with the Council of State, or to a law. This law or decree, when no private property has to be taken, is called an *act of authorization;* if, however, private property has to be condemned, it is called an act *declaratory of public utility.* When resort must be had to a declaration of public utility, the law or the decree which declares that the proposed work is of public utility must be preceded by an inquiry (enquête) of public utility.[2]

The question as to whether the declaration of public utility should be made by law or by decree is decided by the law of July 27, 1870. A decree may suffice when the work, be it a canal, road, railway, etc. . . . is less than 20 kilometers (12.5 miles) long. When the length of the work is not clearly given, although the Administration is not bound by any hard and fast rules, it decides for a law or decree by the character and cost of the work. If it be a question of renewing a work already in existence without making any material change in its former condition, a decree is generally considered as sufficient, but the passage of a law seems necessary when it is a matter of a new work or when the site which it is to occupy has to be radically transformed.

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[1] Public proposals are the rule; informal agreements and day's labor form the exception. The principle of public proposals may only be set aside in certain cases provided for by the decree of November 18, 1882.

[2] The form of this inquiry varies with the nature of the works (see the regulation of February 18, 1834, for works in general; regulation of July 23, 1835, for commercial works; decree of August 15, 1858, for works of defense against floods; decree of August 6, 1881, for tramways and railways of local interest). The inquiry, or better, perhaps, the investigation, consists essentially in depositing, at various points, a set of papers which explain the purposes of the project so as to allow the public to make its remarks. The observations thus collected are examined and weighed, according to circumstances, by a Committee of Investigation (Commission d'enquête), or by a special investigating agent (Commissaire enquêteur).

It has become a general rule during late years in France for the Administration to ask the parties benefited by works for which they have called to contribute toward expenses by which they will benefit. The Administration does not act, however, in accordance with precise rules laid down by any text.[3] But the use of assistance funds (fonds de concours) has become more and more the rule during the last forty years, so that now it may be said that no more works, having new lines of communication as their object, are undertaken unless the State be first assured that an important part of the cost is to be borne by the the interested parties. The rate of one-half of the total cost has been adopted as a rule, still this is by no means absolute.[4]

The interested parties may be sundry persons: departments, communes, chambers of commerce, or even private individuals.

There is no fixed rule about the division of the cost among the different classes of parties in interest.

When the assistance funds amount to a large sum and come from many sources, the Administration requires, as a rule, that they be centralized in the hands of one person who turns them over to the Treasury of the State.

The decree or the law which authorizes the works or declares them to be of public utility, takes note of the offers of assistance which have been made. These offers become, then, real debts, of which the amount can be recovered by legal proceedings, but the interested parties have the right to appeal for relief to the Council of the Prefecture[5] from which the appeal can be carried to the Council of State.

If the assistance funds are to be paid in by an artificial person—that is, by a department, a commune, or a chamber of commerce—this person may be obliged to have recourse to a special authority

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[3]The principle of cooperation by the interested parties may be considered as having been settled by Art. 30 of the law of September 18, 1807.

[4]The Administration now demands that when these assistance funds are paid in they shall be wholly abandoned; long since the idea of loans, with or without interest, has been abandoned.

[5]The Council of the Prefecture is an administrative court which advises the prefect of a department on various legal matters concerning the administration of the affairs of the department. It is the judge in disputes relating to direct taxes and to others which may be assimilated thereto and, very rarely, in disputes relating to indirect taxes; also in matters relating to the work of the Ponts et Chaussées. Its functions are varied; its jurisdiction is limited to its own department. Appeals from its decisions are carried to the Council of State.

in order to form the financial combination which is to obtain the funds. The form of this authority varies, naturally, with the character of the artificial person considered. In order to avoid any miscalculations, the Minister of Public Works is in the habit, before accepting any offer of a large amount of assistance, of obtaining from the Administration which has the oversight of the financial management of the artificial person in question an assurance that this person will be in a position to keep the engagements which he has made.

It may happen that the body which has undertaken to furnish the assistance funds has not in hand the amounts necessary for this purpose. In such a case this body may be authorized to contract a loan and so obtain what is required for the work in view. The authority for such a step is granted in accordance with a procedure and under conditions which vary with the body concerned, and with the effect which the intended loan may have on this body's budget. It is enough, depending on the special case, to refer to the organic texts which govern the financial management of departments, communes, and chambers of commerce. The loans may be made directly by public subscription. This method is rare, however. Generally, the interested parties apply to regular loan establishments, such as the Crédit Lyonnais, Crédit Foncier, etc. The rate of interest varies, naturally, with the time at which the debt matures. This last oscillates, as a rule, between thirty and seventy-five years.

It may be said, as a general rule, that when assistance funds have to be provided, it is the chambers of commerce which obtain the pecuniary resources required. Where this is the case, if it be question of a seaport, the chamber of commerce may levy tolls on shipping at such a rate as to be able to pay the interest on the loan and a certain part of the principal every year. The institution of a right to levy such tolls can be provided perfectly well for the benefit of the departments, communes, or chambers of commerce to reimburse them for the sums which they gave to the State. As a matter of fact, it is the chambers of commerce which profit generally by such tolls as are provided for by the organic law of April 9, 1898, and by the law of April 7, 1902, in relation to the merchant marine.

In principle, however, these tolls can be levied only on ocean navigation. They are determined either on the basis of the gauge (jauge), the number of passengers, or the cargo; this last being

determined in metric tons or in charter tons.[6] These tolls cease automatically so soon as the sums given, which they were to cover, are reimbursed.

The principle of absolute freedom for inland navigation is considered to be an obstacle to collecting tolls. A special law is necessary, therefore, to authorize such tolls. The case came up some years ago in regard to the completion of the canal from the Marne to the Saône and a little later in regard to the Northern Canal (Canal du Nord).

In regard to the first-named canal, the law recognizes that the Chamber of Commerce of Saint-Dizier had offered 5,000,000 francs for the completion of this work; it provides that this sum is to be paid in installments; the first one to be of 500,000 francs paid on July 1 of the year which follows the promulgation of the law, the other payments to be of 750,000 francs each on July 1 and October 1 of the following years, but the Minister of Public Works may put off the date of payment of these installments. The State has to finish the work during the six calendar years following the promulgation of the law; the Chamber of Commerce of Saint-Dizier is authorized to borrow the money, at not to exceed 4 per cent, in any way it chooses and to issue its bonds therefor and to redeem the latter at pleasure; the Chamber of Commerce is authorized to collect tolls during fifty years on a certain specified part of the canal, the period to begin with the complete opening of the canal to navigation, the tolls being per ton-mile but of four different classes: three for freight and one for empty boats, the tolls being applicable over the whole or any part of a certain named section of the canal. The tolls are to cease when the loan is repaid. The toll rates may be reduced under certain specified conditions of increase of traffic. The tolls are collected by the Finance Department, the cost being paid by the Chamber of Commerce of Saint-Dizier on a basis laid down by the Minister of Finance.

In seaports, where advances have been made by corporations, the tolls are levied and collected by the Customs Service.

The question seems natural here: If the work does not succeed because trade refuses to follow the new line, and if the body in question be unable to pay the interest or the principal of the loan, who would have to stand the loss? So far the case has never come

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[6] The metric ton corresponds to a volume of 1 cubic meter; the gauge ton (tonneau de jauge) is 2.83 cubic meters; the charter ton or *tonneau d'affrètement* is 1.44 cubic meters.

up and, apparently, it is not likely to arise. If the Department of Public Works should have any doubt as to the complete solvency of the body, whether department, commune, town, or chamber of commerce, it can obtain full information as to the financial responsibility of the said body from the Department of the Interior. Still, in case of misinformation, it is not likely that such a case should arise because an increase of taxes or, perhaps, an extension of the period of maturity would allow the corporation to meet its obligations, even though a little more slowly. If the supposition of complete failure were to be realized, the loss would have to be borne by the body which had borrowed the funds required to make good the guarantee and which would be obliged to take steps to enter among its obligatory expenses the sums necessary for the payment of the annuities on the loan. In case of necessity, these steps would have to taken ex-officio by the higher Administration.

In other words, the assistance fund becomes a debt to the State, which latter can enforce its claim by law. If funds be borrowed to make the assistance fund good, the lenders have the same recourse against the corporate body (department, commune, town, chamber of commerce, etc.) that they would have against any debtors.

If the works were still under construction when the certain failure of the project was shown, there is no doubt of the higher Administration examining with benevolence the reduction of the original programme, so as to keep within the limits of a combination which could be carried out.

Such, in its broad features, is the way in which public works are started, studied, and carried out in France when the work is done directly by the State. When taken up by the Government, all the work required by the project is studied and supervised by the engineers of the Ponts et Chaussées. The principle of an assistance fund, furnished by parties desiring the improvement, is now fully adopted; this fund is set usually at fifty per cent of the total cost of new works. In the case of new works to extend old ones, this percentage may be reduced. In some cases, when the carrying out of the work has been very earnestly desired and the completion of the operations is to be hurried forward, the corporate body may offer a larger fund. This was done at Dieppe some years ago when the city offered as much as 85 per cent in order to have the work hurried forward.

Hydraulic Dredges and Dredging on the Improvement of the Upper Mississippi River

BY

Mr. JAMES DICK DU SHANE
Assistant Engineer

The hydraulic dredges on this work are used in connection with the improvement by contraction works of the low water channel of the Mississippi River from St. Paul to the mouth of the Missouri River, under the direction of the United States Engineer Office, Rock Island, Ill. They are all of the same type, but vary somewhat in size and details of machinery. There are five, all of which are the property of the United States, and three more are in process of construction at the shops of the Des Moines Rapids Canal, Keokuk, Iowa, to be completed during the year 1912.

The first hydraulic dredge operated successfully on this part of the river in improving the channel was the *Geyser*. The first practical channel work with it was done in 1894, between St. Paul and Lake Pepin. The dredge, as originally built, was defective for handling in operating, and was remodeled by the writer during the following winter; thereafter, it became the type for all the hydraulic dredges built by the Rock Island Office. The distinctive feature of the hydraulic dredge, is, of course, the pump. The distinctive feature of this hydraulic dredge, and one that makes it a type, is the catamaran ponton, attached at mid-length of one side of the dredge by a swinging joint, for carrying the suction pipe.

The three dredges last built, 1908-1909, and the three in process of building are practically alike, except as to boilers and size of main engines and pumps, two being 15-inch and four 18-inch dredges. One has two 125-horsepower Bonson boilers, one has two 125-horsepower Hawkes boilers, and the four 18-inch dredges have, or will have, a battery of three boilers, 42-inch diameter, 22 feet long, of the Mississippi River type. All have cross-compound condensing engines; the Ideal engines on the smaller dredges are of 225 nominal horsepower, furnished by A. L. Ide & Sons, Springfield, Ill.; those on the larger dredges are Erie-Ball engines of 300

nominal horsepower, furnished by the Ball Engine Company, Erie, Pa. The latter engines are 13-inch diameter for the high-pressure cylinder, 26 inches for the low-pressure cylinder, and 16 inches stroke, to operate at about 200 revolutions per minute. They have the Armstrong governor, as have the Ideal engines, which will be described later. The larger dredges have a hull with steel frame, planked on the bottom with oak and on the sides with fir; the decks are fir with concrete foundations for engine, pump, and boilers. The discharge and suction pontons are of wood.

A description of one dredging plant will answer for all. The dredge *Pelee* will be selected for description, because the writer is more familiar with it. This plant consists of one dredge, one quarter-boat, two coal barges, twelve discharge pontons, and the suction ponton. (A small tow-boat should accompany each dredge as tender, when the dredging operations are not within the range of a tow-boat connected with dam construction.) The *Pelee* plant was built at the United States boatyard, South Stillwater, Minn., 1908-1909. The machinery is all of standard makes and commercial sizes. The *Pelee's* hull is of Douglas fir, all except the bottom planks being treated with creosote by the open-tank method. The hull is 119 feet long, 30 feet wide, and 5 feet deep. In shape it is like an ordinary barge, having flat bottom and a rake at each end. To give longitudinal stiffness to the hull, the gunwales and bulkheads are made of timbers 6 inches thick, set on edge, and drift-bolted every 10 inches with $\frac{3}{4}$-inch round steel, there being seven bulkheads extending the full length of the hull. To stiffen the hull transversely, and to assist in distributing the weight of the machinery, there are eight Howe trusses, extending through the bulkheads and having the ends fastened to the gunwales. To assist further in distributing the weight of the boiler, truss rods are passed under the lower chords of the two cross trusses under the boilers, the rods taking hold at the top of the bulkheads out from each end of the boilers. The foundation of the main engine consists of a platform of 8 by 10 inch timbers, laid on edge, extending across the bulkheads; this platform forms part of the deck. Below this platform timbers, extending through the bulkheads near the bottom, take the bolts for fastening down the engine base. The foundation is intended to distribute the weight of the engine and to take up the vibration of the engine and transfer it to a larger area of hull. The bottom and deck of the hull are made of 3-inch fir planks surfaced to $2\frac{1}{2}$ inches thick, placed

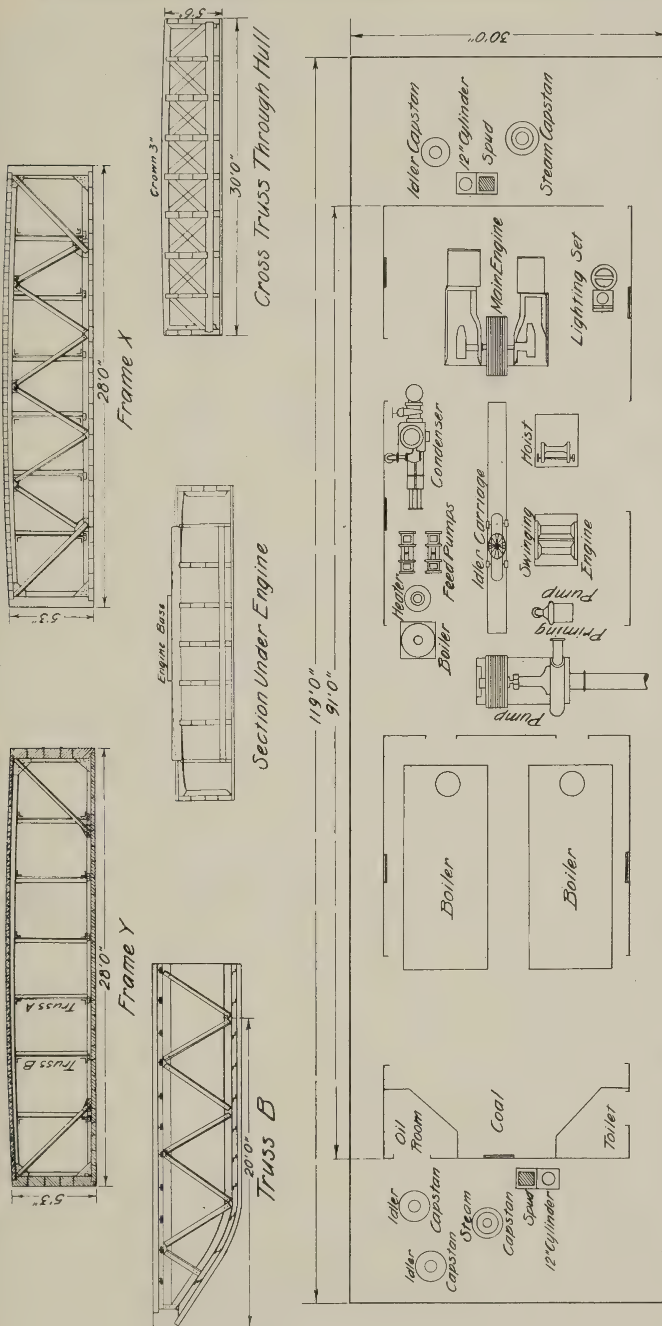


Fig. 1. Longitudinal truss B; Frames X and Y pertain to the composite hull of the dredge *Etna*. Remaining plan and sections pertain to the dredge *Pelee*.

cross-wise of the hull. One spud is placed at each end of the dredge to assist in holding it in position when dredging, and to aid in handling it during the operation of making a cut through a bar. The cabin is a one-story structure, increased in height over the boilers.

The machinery consists of two boilers, one main engine, one sand pump, one priming pump, two boiler feed pumps, one hoisting engine, one swinging engine, an electric generator set, spud hoists, etc.

The boilers are of the Hawkes pattern. This is a type of water-tube boiler, with two rows of 4-inch tubes over the furnace connected to the horizontal shell by a water leg at each end of the boiler. The shell is 60 inches diameter, 16 feet long, and contains sixty-eight tubes, 3 inches diameter; the lower row of water tubes consists of seven, encased with tiles over the furnace and for some distance beyond the bridge wall; the upper row contains thirteen tubes, having a top covering of tiles. The flame and gases pass from the furnace back toward the rear end of the boiler, up between the lower tubes, return forward under the upper tubes and then pass through the tubes in the shell to the stack. The furnace is much like the usual type on river boats—that is, of fire-brick construction surrounded by a sheet-iron jacket. The furnace is provided with shaker grates. On each boiler is a 60-inch exhaust fan direct connected to a 5 by 5 inch engine for increasing the draft in the furnace, the chimneys not being intended to produce the requisite draft. The capacity of each boiler is 125 nominal horsepower, but considerably more steam than the rated capacity can be generated. These boilers are intended to carry 150 pounds of working steam pressure. A donkey boiler, 3 feet diameter, 8 feet high, is provided for assistance in washing the main boilers and to supply steam to any of the auxiliary engines that may be needed at times when the main boilers have no steam. The auxiliary boiler carries 100 pounds steam pressure. The main steam pipe is 5 inches diameter and extends between the boilers and the main engine only. The auxiliary engines are supplied from a separate header, containing an automatic reducing valve near the boilers to reduce the steam pressure from 150 pounds at the boilers to 100 pounds in the header for use in the auxiliary engines. The small boiler is connected with this header. There are two duplex boiler feed pumps, 6 by 3½ by 6 inches, the feed water passing to the boilers through a Wainwright 400-horsepower vertical

heater, which receives the steam from all the auxiliary engines and exhausts it to the atmosphere. These pumps also furnish water to the runner bearing of the dredging pump to protect it from sand, and also to the fan engines to cool the cylinders; they are so connected with the pipe system that one or the other may be used on either service.

The main engine is an Ideal horizontal, cross-compound, condensing, high-speed engine, the cylinders being 12 and 24 inches diameter and 14 inches stroke with piston valve on the high-pressure cylinder and balanced slide valve on the low-pressure

Fig. 2. Dredge *Pelee* (No. 3) making a cut in the channel and depositing the sand between dams.

cylinder. There is a receiver between the cylinders. An automatic governor attached to the flywheel controls the speed within a variation of about 4 per cent. The engine is capable of running at speeds up to 275 revolutions per minute, but in actual dredging operations it is run at speeds around 260 R. P. M., at which speed with a steam pressure in the boiler of 150 pounds, when pumping sand, the engine will develop about 300 indicated horsepower. The engine was well balanced in making, and causes but slight vibration in the vertical direction and none horizontally. The con-

denser is of the jet type, 8-inch steam cylinder, 14-inch air cylinder, and 12-inch stroke. The vacuum produced is about 25 inches of mercury. The usual steam and vacuum gauges are provided on the engine.

The dredging pump is a Morris 15-inch sand pump connected with the engine by means of a rope drive of the American type, the endless rope of 1⅜-inch diameter running over 10-groove pulleys, that on the pump shaft being 62 inch and on the engine 66-inch diameter. A tension carriage, carrying an idle rope sheave, runs on rails between the engine and pump, and takes up the slack in starting and stopping the pump and provides for any stretching of the transmission rope. The pump casing is about 2 inches thick around the volute; the sides of the casing are provided with cast-iron liners of the same diameter as the pump runner. The open runner, called also the piston, is used in the *Pelee's* pump; it is 51 inches diameter and has blades of steel, 11 inches wide, bolted to the arms of the cast-iron hub. Pressure and vacuum gauges are attached to the discharge and suction pipes near the pump to indicate the discharge and suction operation, as a guide to the operator for regulating the inflow and outflow while pumping. The suction pipe is 18 inches diameter, and the discharge pipe 17 inches, both connected to the pump by reducers. The discharge pipe is carried from the pump overhead to each end of the dredge, where it descends by means of a reverse curve to the height of the ponton line. An elbow at the pump outlet can be turned and attached to the pipe line for discharging in either direction. This is desirable, so as to permit depositing the dredged material on either side of the river, as may be required. The suction pipe is 80 feet long and the discharge line is variable in length up to about 1,000 feet, of which about 640 feet are carried by the dredge and discharge pipe pontons, the remainder being put out on a bar or on shore when necessary.

The main pump is primed by a 6-inch centrifugal pump, direct-connected to a 7 by 8 inch engine, capable of raising 1,000 gallons of water a minute against 30 feet of head. This pump is itself primed by a steam syphon. The engine for hoisting and lowering the suction pipe is an 8¼ by 10 inch double-cylinder hoisting engine with single drum, around which the hoisting cable winds. The engine for swinging the suction ponton is a 6 by 7 inch double-cylinder Clinton midship nigger with two drums, around which wind the cables attached to the outer end of the suction ponton.

The spud hoist consists of a steam cylinder 12 inches diameter by 8 feet long, set vertically alongside the spud; the piston rod carries a cam clutch at the upper end, which, as it begins to rise, grips the spud, raising the latter as required up to the maximum lift of 6 feet. If higher lift is necessary, a pawl is thrown into a rack on the spud to sustain it while the piston is being lowered to take a new hold. The piston is raised by admitting steam to the lower end of the cylinder and lowered by exhausting it through a simple three-way valve. The spuds are of oak, 14 inches square by 34 feet long, having a cast-iron pointed shoe at the lower end. Steam capstans having 5 by 7 inch double engines, supplemented with idler capstans, are provided on each end of the dredge for moving the dredge during operation. The electric lighting plant consists of a direct-connected 8-kilowatt generating set, the dredge and pontons being wired for incandescent lights. The disposition of the machinery on the floor plan is such as to afford the greatest convenience in handling. A set of hand tools is provided for making all ordinary repairs during the working season.

The catamaran suction ponton, which is the distinguishing feature of this type of dredge, is composed of two small barges, 56 feet long, 6 feet wide, and 34 inches high, with raking sides and forward ends; these barges are fastened apart at both ends, leaving an opening between 50 feet long and 4 feet wide, through which the suction pipe is operated. This ponton is provided with frames at each end for carrying the suction pipe, and sheaves and cable for raising and lowering it. The suction ponton is connected with the dredge by means of a frame hinged to the dredge at the middle of one side, the other end of the frame is connected with the rear end of the ponton by a pivot joint. The ponton thus has freedom of vertical and horizontal motion, swinging about the pivot as a center in a half circle. The swinging motion is given by wire cables attached to the forward end of the ponton and running over sheaves at each end of the dredge to the swinging engine. The suction head on the *Pelee* is simply the lower section of the pipe squared up a little larger in area than the suction pipe, with wing valves, for use in priming, hinged at the sides instead of at the middle of the opening.

The pontons for carrying the discharge pipe line are all small barges, 36 feet by 12 feet by 2 feet, 3-inch gunwales, with raking sides and ends. There are twelve of these. An A-frame at each end serves to connect them together and to the end of the dredge,

by pivot joints, so as to form a flexible line about 600 feet in length. The discharge pipes are carried in saddles on deck of the pontons, the pipes being joined together by means of rubber sleeves over the pivot connection of the ponton. About 400 feet of pipe, in 20-foot lengths, is provided for use on shore at the end of the floating line. The suction pipe is made of No. 6 steel; the discharge pipes on pontons are of No. 8 steel; the part for use on shore is of No. 16 steel, in order to lighten the weight for handling; the latter are connected by simply slipping one end of section line into another, much like a stove-pipe joint.

Instead of providing quarters on the dredge for the crew a separate quarter-boat is used. The latter arrangement is better because of less cabin on the dredge to catch wind, which would interfere more or less at times with operating the dredge, and also for the reason that it gives a quieter and better place for taking care of the crew. The quarter-boat is simply a two-story cabin, 49 by 18 feet in plan, on a barge 68 by 22 feet by 3-foot gunwales. This is outfitted for a double crew, consisting of eighteen men.

The coal flats, of which there are two, are barges 68 by 16 feet by 4-foot 6-inch gunwales, with deck set down 17 inches. Larger barges are used with some of the other dredges.

The dredges are to be operated in connection with the regulation of the river from St. Paul to the Missouri River by contraction work. This stretch of river is 658 miles long, and has a low water slope of about 0.5 foot per mile. The river valley on the upper portion is from 2 to 3 miles wide; on the lower portion, 5 to 8 miles. The banks are low, and on the upper half the valley contains many sloughs, which serve as high waterways, but at low water are frequently dry or contain little water. The river carries little silt, but may be described as a sand-rolling stream. High water usually appears in May and June; low water begins, usually, the latter part of July or in August and continues until the following spring; generally there is a slight rise during September or October. There are exceptional periods when the river may be low during the spring and high during the autumn. Navigation is suspended during the winter, when the river is covered with ice. The stream is broken by two rock gorges which form the Rock Island and the Des Moines rapids, at both of which places the river bottom is rock. Excepting these two places the river flows over a bed of sand, grading into small gravel at some localities, and at a few places containing deposits of glacial drift, con-

sisting of gravel and boulders. The river contains many islands, but the sloughs back of them have generally been closed by dams, so that the river is now practically confined to one main course at stages lower than 4 feet on the upper portion, and 6 feet on the lower.

In addition to the sand already in the main course, much is brought in during floods by the larger tributaries, and much is added to the channel each year by cutting banks not yet revetted. Bank erosion will be entirely overcome within a few years by revet-

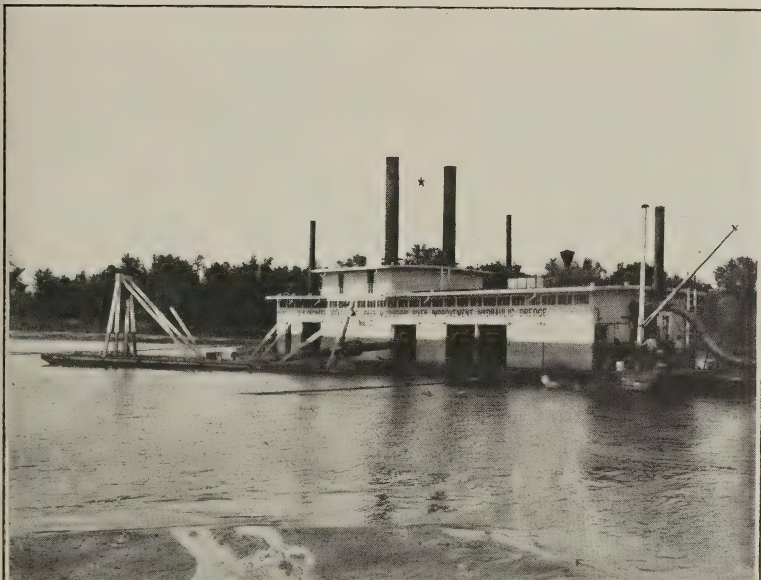


Fig. 3. Beginning to raise the spud on one end of the dredge preparatory to advancing that end in the cut.

ing them; contributions from side streams will continue, unless these in time become regulated.

Hydraulic dredging operations on this stretch of the river are intended to supplement the regulation works, and are performed for several different purposes.

Theoretically, a regulated river should be capable of keeping itself in good condition and should be able to remove the sand brought down during the high water. Practically, this is not altogether the case, since much of the sand moved by high water will be deposited farther down on the crossings—that is, where the

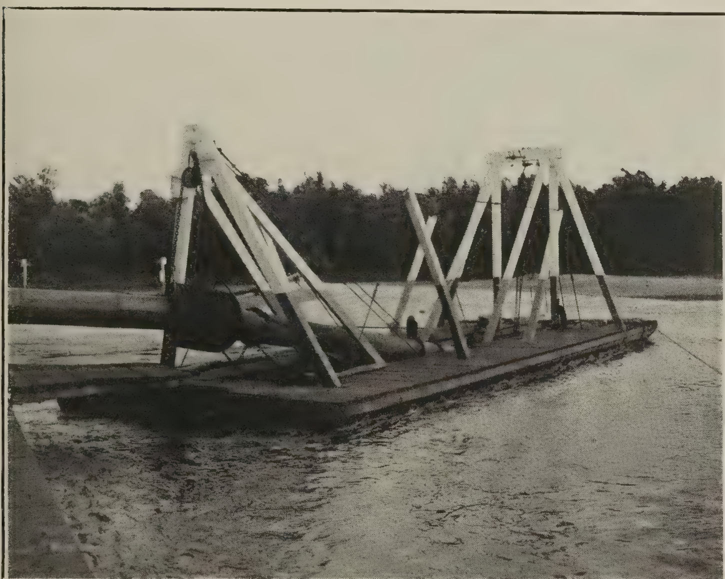
river passes from the pool along one bank to that along the other. Here the current at times will not follow parallel with the direction of the channel as planned, but in crossing will take a diagonal direction, thus the real cross-section of the flow will be lengthened, the velocity of the current will be diminished and, as a consequence, sand will be deposited and the crossing shoaled. These are the places where navigation meets trouble during low water. While there are several hundred crossings in this stretch of the river, any one of which may become shoal, comparatively few are shoal enough to become troublesome during any one season, and as the regulation works progress toward completion these troublesome bars become fewer. But there will probably always be some crossings in particular localities where dredging may be required from time to time, though not usually in successive years.

The bars where navigation may encounter hindrance at low water are known before the low water season is at hand, and preparations for dredging can be made far enough in advance to complete the cut before the stage falls sufficiently to render them obstructive. In such cases the cut should be made deep and wide, in order to remove a considerable portion of the sand from the channel, as well as to make the improvement more durable. During seasons of low water if shoal bars are numerous in any particular section of the river shallow cuts, in the nature of emergency work, may be made, the expectation being that the cuts will remain open only during the low water period. With the dredging equipment now on hand and nearing completion it may be expected that navigation on the Upper Mississippi will not be interrupted hereafter by obstructive bars.

Dredging on the crossings becomes of immediate use to navigation by opening up an obstructive place between long pools of deep water. The dredged cut on these crossings is usually laid in the direction of the current; this enables the river to assist in keeping the cut free from inflowing sand during the progress of the work and in maintaining it after completion. Generally, the dredged channel will remain open throughout the season, and frequently the improvement will continue for a number of years. The secret of success of the dredging of crossings lies in selecting the proper location and direction of the cut to be made.

Dredging is sometimes necessary in connection with the regulation of the river at places other than the regular crossings. Sand will be moved down into pools and form a bar between the shore

ınd the ends of the dams on the opposite side of the channel way. While the bar is passing through this pool, the alignment of flow vill be changed and two crossings may be created in a section of the iver where contractive works are already completed. In order o correct this condition, the bar should be removed as soon as practicable by dredging. In some cases it becomes desirable to remove practically the entire bar by dredging, depositing the maerial either on the low banks or in adjacent sloughs, or between he dams as near the opposite shore as possible. In other cases, vhere the direction of the reef is such that the bar will naturally



'ig. 4. Suction ponton, showing means of supporting and raising and lowering the suction pipe; also showing the flexible joint in suction pipe and the means of joining the dredge and ponton so as to permit swinging the latter. The suction pipe is lowered for dredging, and the ponton is in the act of swinging across the cut.

ravel toward the dams, it will usually be sufficient to make a cut hrough the bar, leaving, perhaps, the larger portion to be moved y the current toward the dams, eventually to be swept at high vater entirely out of the channel, lodging between the dams.

At times, dredging becomes desirable for the purpose of diverting the channel to another position in order that contraction works nay be constructed and a proper direction be given to the regulated river. In such cases, it may happen that advantage can be

taken of the dredging to deposit the dredged material in deep water on line of the dams to be built, thus saving brush and rock in the construction of the dams and decreasing their cost. In other cases, dredging becomes essential in order to obtain a proper section of waterway and to prevent the sand from being driven downstream, where it may influence unfavorably either regulated or unregulated sections.

Dredging may also be undertaken for the sole purpose of furnishing material for dam building. It often happens that new sloughs are created by high water, cutting islands in two and making depressions behind revetments. These can be closed or filled by dredging. The sand may be taken from the channel, thus causing a direct improvement therein, or it may be taken from a location more convenient for handling the dredge and depositing the sand in the dam. A number of sand dams, finished with one course of brush and rock laid on top and a protective apron around the outer end, were built some fifteen years ago on the upper portion of the river, and have proven to be durable, satisfactory, and economical in construction and maintenance. Sand may also be deposited near a cut that is being dredged to form a temporary dam and confine the water and assist in the improvement by increasing the force of the current for scouring and carrying away the sand from the cut.

It may also become desirable to resort to dredging, in order to prepare the foundation for dams or sills. No dredging for such a purpose has thus far been undertaken on the river, but there are no doubt numerous places where it might have been advantageous, had a dredge been available, especially in cases where closing dams have been built across shallow sloughs.

Generally, dredging will be confined to crossings for some years to come; but, as conditions on the crossings permit, much dredging will be necessary to remove bars which gradually accumulate in the channel in sections of the river where the works of contraction are partly or entirely completed. In fact, the dredges should be used for removing from the channel and depositing between dams large quantities of sand, particularly at localities where construction of regulation work is in progress, thus avoiding the frequent necessity of making shoal places shoaler during the early stages of the movement of the sand due to increased velocity of current, by reason of the contraction by dams, and also avoiding driving this sand farther down into a part of the river improved

or to be improved later, thereby creating a new disturbance. The sand should be removed directly from the channel and deposited where it will remain out of the way for all time, regardless of the fact that the contraction work will accomplish the same object, though requiring much more time because of the limited periods of high water, when, only, the sand can be moved by the action of the current to a resting place between the dams. As the river becomes more nearly regulated in any portion by building the contraction works out to the channel limits adopted, greater care in dredging will be necessary in order to limit dredging operations

Fig. 5. The out end of the suction ponton, showing the hoisting frame and the suction head. The cable attached to the corner of the ponton is one of the swinging cables. The chain seen hanging over the side of the ponton is attached to the suction head and is used to hold the head at grade when dredging and to pull the suction pipe as the ponton swings; a similar chain is used on the other side.

to the needs of navigation and to secure a cross section suited to the flow of water.

The plan of operation in dredging for channel improvement is simple. A certain location is decided upon and a survey made, having which and being familiar with the local conditions, a position is selected for dredging. The lines of the cuts are marked either by flags on shore or by buoys in the water. The dredge is

brought approximately into position for beginning the work. If the river is narrow, so that the lines for holding the dredge in position and moving it during the operation can be fastened on shore and to anchors placed on dams, then these are used; if the river is wide disc anchors jetted into the sand bed of the river are used. The disc anchors are, in fact, not discs but flat truncated cones of cast-iron, about 14 to 16 inches diameter at the base; one end of a wire cable, about 50 feet long, is fastened to the disc, the other end of the cable having an eye into which is fastened the holding line and a small line attached to a buoy to mark the location of the disc. The disc anchor is jetted into the sand to a depth of about 9 feet, usually by the steamboat attending the dredge. The holding lines, which are manila rope 1,200 feet long, lead to the capstans on each end of the dredge. Breast lines are sometimes necessary to prevent the dredge from swinging out of the cut, and to haul back when the dredge moves out for passing boats. These being in readiness, the dredge is swung into position lengthwise across the current and a cut from 90 to 100 feet in width is begun. This cut may be made either upstream or downstream, generally it progresses upstream, for the reason that when dredging downstream in shoal water loose sand stirred up by the suction head will move down and form a ridge in advance of the dredging, thus making a shoal place still shoaler. One cut being finished, other cuts are made alongside if necessary to secure the width of channel or improvement desired. In moving the dredge in the cut it is advanced one end at a time about 3 or 4 feet, by raising one spud and hauling in on the holding line at that end, the other end swinging on the spud as a pivot at that end. The suction ponton is then swept over a half circle by operating the swinging lines. The operation is repeated for advancing the other end of the dredge. While the ponton is swinging the pump is running and the suction head moves on the river bottom, being held at the proper depth by the hoisting cable or by chains attached to the head and fastened to slotted holding-bars at the sides of the ponton. The sand, mixed with water in proper proportion to make transportation practicable, is conveyed through the suction and discharge pipes and deposited where desired, usually outside of the channel limits, generally in places where it will be retained and prevented from getting back into the channel. The dredge advances in the cut at a rate of from 15 to 30 feet per hour, depending on the depth of cutting and the character of material being

dredged; where there is much gravel in the bar less material will be dredged and advance will be slower. In former years it was thought necessary to have a jet at the intake to loosen the sand so that it might be readily picked up by suction of the pump. Jets are no longer used, the suction being sufficiently strong to draw into the suction head all the load that can be passed through the discharge pipe.

To produce the necessary suction and pressure for picking up the sand and transporting a sufficient load, the pump should be in good condition and run at proper speed. The speed of the pump should be such as to secure a high velocity in the suction and discharge pipes. The velocity in the pipe line should be not less than 15 feet per second. To allow for variation in the length and elevation of the discharge, as well as of the load, the pump engine should be provided with variable speed governor, which can be adjusted for any speed desired within the range of the engine. The amount of work done increases rapidly as the speed increases, and to get the greatest capacity or maximum output of the dredge the pump should be run at as high speed as practicable. It has not been observed that any of the dredge pumps have reached the limit of speed.

The runner may be either the closed or open type; there seems to be little difference in their efficiency. The closed runner produces in operation a strong end pull toward the suction side of the pump; it has been difficult to provide a bearing that will take care of this pull properly; the bearings so far used have proven very inefficient. For this reason an open runner was devised during the early life of the *Geyser* on this end of the river, in order to abate this difficulty. This type has proven successful in completely overcoming the end pull. Another advantage of the open runner is that provision can be made for keeping the clearance between the runner and the casing small, which is essential for good work. With the open runner cast steel shoes are sometimes used to retard the wearing of the runner by friction of the sand between the runner and the rear liner, where the greatest wearing occurs. It is thought by the writer that the open runner with five blades is best for use on the dredges under consideration. It runs perfectly balanced as to end pull, and it will do more work than a runner with a less number of blades and, probably, as much as a runner with a larger number. As between the shrouded, or closed, runner and the open type there is little difference in

efficiency so long as the clearance between the runner and the casing is kept small.

The suction, or intake head, should be of such form and strength as to stand the rough usage to which it is put on the river bed. The size of clear opening should be little, if any, greater than the area of the suction pipe. A suction head having the opening full size is preferable—that is, one which is not obstructed by bars over the entrance to prevent stones and drift from getting into the pipe, and which has the priming valve hinged at the sides instead of across the middle of the opening. It is desirable to pass all stuff that will go through the pipes and the pump, so as to avoid frequent stoppages of the pump in order to clear the suction head of drift or stones; at best, the stoppages are frequent.

There should be no air leaks in the suction pipe or pump. A very small quantity of air leaking into the pump will cause a large decrease in the output. So far as known, no experiments have been made on dredges as to air leakage, but the effect of air leakage on any dredge where it occurs is very noticeable in the quantity of sand pumped. An air leakage of 3 or 4 per cent of the volume of water pumped will decrease the output of sand probably as much as 30 or 40 per cent. It is, therefore, most desirable that air leakage be prevented.

No extended tests of the dredges have been made for efficiency or capacity. It is thought that the engine efficiency is about 90 per cent and that the efficiency of the engine and pump varies on the different dredges between 50 and 60 per cent. The capacity of a 15-inch dredge, everything being in good working order and pumping through the ordinary length of pipe line—that is, 500 to 600 feet, should be about 300 cubic yards per hour, and for the 18-inch dredges about 400 cubic yards; this is for actual time of pumping sand. The yardage actually pumped per hour of working time of dredge is considerably smaller, because of the fact that during much of the working time the pump is either stopped for various reasons or, if running, may be pumping water, the suction head having been raised out of the sand. About one-third of the working time is lost in stops for handling pipe line, for moving out of the way of passing boats, and for delays in moving in the cut. Some part of the actual running time of the pump is consumed in pumping the water alone, from causes not requiring the engine to be stopped; these periods are short, from a fraction of a minute to several minutes, and are frequent. The actual time of pumping

sand is probably between one-half and two-thirds of the working time.

The cost of a 15-inch dredging plant complete for work, excepting an accompanying steamboat to act as tender, is about $40,000. For 18-inch plant the cost is about $50,000, exclusive of the tender. The cost of operating the larger dredge plant is not much greater than for the smaller, the crew being practically the same size; the greatest difference in cost of operating will be due to greater consumption of coal of the larger dredge.

The cost per cubic yard dredged depends upon the purpose for which dredging is done, the character of material and the length of discharge line, as well as in a large measure upon the distance that the dredging is from the coal field, as the cost of fuel is a considerable part of the operating cost. Owing to the unsatisfactory method of measurement heretofore used, namely, the percentage method, the cost account of the material dredged has not been exact. Using this method of measurement, the cost has been estimated at from 4 to 6 cents per cubic yard for the different dredges at various localities. It is known that the percentage method does not show a correct output, and it is believed that the cost per cubic yard, including office expenses, and depreciation of plant, should be about double the figures stated above. The method of measurement used, experimentally, during 1911, will give a nearer approximation to the quantity actually dredged, namely, taking soundings in advance and the wake of the dredge from which to compute daily the quantity dredged.

The dredging crew consists usually of eighteen men, working in two shifts of ten hours each. The time of operation with double crew may be continuous for the twenty hours, or the dredge may be stopped during meal times. When running with a single crew, the crew consists usually of twelve men, working ten hours per day. Occasionally it may happen that additional men may be required, especially if a long pipe line is used on shore, or across bars, in which case some laborers would be hired for the time being to help on this part of the work. Subsistence and quarters are provided for the crew on a quarter-boat accompanying the dredge.

Practical Determination of the Magnifying Power of Telescopes

BY

Lieut. WILLIAM F. ENDRESS
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These methods are sufficiently exact and require no elaborate apparatus for their use, nor knowledge as to the refraction index of the lens material, curvature, or exact position of the optical center. For all practical purposes in solving lenses, if one face is plane and the other curved, one end of each measurement made to the lens should be at the center of the curved face and at the surface, as the optical center lies there. In double concave or double convex lenses, the optical center may be assumed to be at the geometrical center. This holds both for simple and compound lenses.

The following method may be used for any form of refracting telescope, except the Galilean; it may, therefore, be used for all forms of surveying instruments: Focus clearly on some distant object and then point the telescope to the sky. Hold the eye a few inches from the eye-piece and look through the telescope. Move a card, pencil, or other well-defined object across the field of view and note its position on the object-glass when it is just at the edge of the field of view. Measure from this point to the edge of the object-glass as seen from the front of the instrument. Twice this distance subtracted from the visible diameter of the object glass is the true aperture. It differs from the apparent aperture (diameter of the object-glass) due to the interposition of a diaphragm to improve the definition.

Now hold a piece of translucent material against the eye-piece and measure the diameter of the bright circle of light seen thereon. This is the diameter of the emergent pencil of rays.

The diameter of the true aperture divided by the diameter of the emergent pencil is the magnifying power of the telescope. This method will not answer for telescopes of the Galilean type as, in that case, the emergent pencil is divergent.

To determine whether or not a field glass or telescope is of the Galilean type, examine the eye-piece. If of the Galilean type, one or the other face of the eye-piece (always simple) will be concave.

The following method is applicable to all instruments used in surveying, as well as field glasses and opera glasses of the Galilean type:

Measure the angle subtended by some clearly defined distant object by means of a transit instrument. The angle should preferably be between two and four degrees. Let this angle be called A. Now take the instrument, whose magnifying power is to be determined, and focus it on the same object. Reverse it and place it in front of the object-glass of the transit, its object-glass facing that of the transit. Again measure the angle subtended by the object, but this time as seen through the reversed unknown instrument. Call the angle so obtained a. Having the two angles,

$$\text{Magnifying power} = \frac{\tan \frac{1}{2} A}{\tan \frac{1}{2} a}.$$

If there is no transit instrument available to use in the determination of Galilean field-glass power, it becomes necessary to obtain the result by analysis. This is done by determining the characteristics of the object-glass and the eye-piece separately and then combining the results.

For all cases,

$$\text{Magnifying power} = \frac{\text{focal length of object-glass}}{\text{focal length of eye-piece}}.$$

TO FIND THE FOCAL LENGTH OF THE OBJECT-GLASS.

Use the lens as a burning-glass with the sun and move a screen back and forth until the image is brightest and clearest. Measure the distance to the lens (optical center) and the result is the focal length. Also,

Use any source of light (a piece of tracing cloth on which is drawn a large black circle and behind which there is a light will serve) and obtain, through the lens, a perfect image on a screen. Measure the distance from the source to the optical center of the lens and call this p. Also measure from the optical center of the lens to the screen and call this distance p'. If the focal distance of the lens be F, then

$$\frac{1}{p} + \frac{1}{p'} = \frac{1}{F}.$$

It is seen that the first method is the limiting result of the second, since the sun may be assumed at an infinite distance, in which case

$$\frac{1}{p}=0.$$

TO FIND THE FOCAL LENGTH OF THE EYE-PIECE.

This lens is always concave and hence the emergent rays diverge and no true image can be obtained, but only a virtual image. However, an image sufficiently well defined may be obtained, and the focal length of the lens determined as follows:

Cover the eye side of the eye-piece with lampblack or other opaque substance. Lampblack may be smoked on with a candle without injuring the lens, if care is used. Set a pair of dividers at some certain unit about half the diameter of the eye-piece lens, and with the two points carefully make two very small holes in the coating, equidistant from the center and on a diameter, the distance between them being the distance between divider points. Place a shield around the lens to prevent the passage of all light beyond the lens, except that which passes through the holes in the coating. Allow sunlight to shine on the lens and two spots of light, blurred on the edges, will be seen on the screen back of the lens. Double the distance between divider points, and move the screen until the distance between the centers of the spots of light is double that between the holes in the coating. The distance from the optical center of the lens to the screen is then the focal distance of the lens. For this method either direct sunlight must be used, or else the screen must be shaded as in focusing a camera, for the amount of light passing through the holes in the coating is very small.

Another method is to use a convex lens of known focal length in combination with the concave lens. In this case, the convex lens must have a very short focal length in order to render the combination convergent by overcoming the divergent characteristics of the concave lens. Place the two lenses close together and let d be the distance between their optical centers. Determine the focal length of the combination by means of one of the methods employed for the object-glass and let it be called F. Then, if f be the focal length of the unknown, and f' that of the known lens, we have,

$$\frac{1}{F}=\frac{1}{f'-d}-\frac{1}{f}$$

from which f may be determined.

The methods above given for determining magnifying power,

while not precise, are far more exact than the methods of comparison in which the observer looks through the telescope with one eye and compares the image so seen with its size as seen with the naked eye at the same time. The comparison method is rough, requires considerable practice, and can not be relied on. Due to the limitations of human vision, it can not be used for far-seeing instruments. The second method given above is, provided a transit is available, far more accurate and but little more trouble.

The following table shows the results obtained by these methods

Fig. 1. Method by angle-measurement.

and their comparison with certified magnification by the Bureau of Standards, Department of Commerce and Labor.

Methods.

Glass.	Maker's Rating.	Bureau.	Lens Analysis.	Angles by Transit.	Measurement of Apertures.	Eye Comparison
Lemaire No. 1		5.1	4.8	4.8		4.6
Lemaire No. 2			4.5	4.5		4.6
Bardou		6.6	6.4	6.4		6.3
K. & E. No. 6931	8	6.8	6.9	6.7		6.6
K. & E. No. 6932	9	7.5	7.5	7.4		7.0
Signal Corps, Day	6.5	6.0	6.0	5.9		5.7
Signal Corps, Night	4.5	3.8	3.9	3.7		3.6
Busch	10.0	10.1		9.9	10.3	10.0
B. & L.	10.0	10.5		10.5	10.5	10.5

The Survey of Pemba*

BY

Capt. J. E. E. CRASTER
Royal Engineers

Pemba is a coral island some 400 square miles in area, lying 60 miles north of Zanzibar, and forming part of the Zanzibar Protectorate. Upon it is grown more than half the world's total supply of cloves; a very valuable crop, upon which the Zanzibar Government levies an export duty of 25 per cent. The clove plantations, which cover about a quarter of the island, are owned by Arabs, who until the abolition of slavery in 1897 employed nothing but slave labor. It is said that in the flourishing days of the slave trade 14,000 slaves a year were imported into Zanzibar and Pemba, of whom the majority died within a year or two.

The natives of Pemba are closely allied to the Swahilis of Zanzibar, and speak a dialect of the Swahili language. There are also in the island a considerable number of freed slaves, who belong to nearly every tribe in East and Central Africa.

CLIMATE.

The temperature varies but little throughout the year. The minimum cold weather temperature at night is about 74° and the maximum day temperature in the hot weather 90°. Heavy rain falls throughout April and May, and there is a second rainy season in November. Showers are of almost daily occurrence during the other months of the year, and the air is always saturated with moisture.

PRELIMINARY RECONNAISSANCE.

In August, 1910, Pemba was visited by Major Gordon, R. E., who made a report, and an estimate for the survey. In April, 1911, a survey party consisting of Captain Craster, R. E., Lieutenant Kyngdon, R. G. A., Lance-Corporals Cager and Whitters, R. E., left England, and arrived in Pemba at the end of May.

MEASUREMENT OF A BASE.

The island is about 40 miles long, and 6 to 12 miles wide. Some 2 or 3 miles off the western coast lies a chain of small islands, and on the eastern shore of the largest of these a base had been selected

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by Major Gordon. Nearly the whole length of the base lay below high water mark, and the central portion below the low water mark of neap tides, so that the measurements could only be made at low water during spring tides. This involved a delay of ten days between the first and second measurements. The measuring party were often working in water a foot or more in depth, but this caused little difficulty and had the advantage of keeping the tape at a uniform temperature.

The base was measured with a 300-foot steel tape, and the end of each tape length was marked by a steel toilet pin stuck in the side of a wooden picket, driven firmly into the ground. The tape was stretched with a spring balance to a tension of 20 pounds. This is as heavy a pull as can be conveniently applied by hand. If tapes longer than 300 feet are used, a heavier pull is required, and it is desirable that some mechanical apparatus should be provided for applying it.

The first measurement of the base, after making the necessary corrections for temperature, etc., gave a value of 13,817.343 feet, and the second a value of 13,817.232 feet. The probable error was therefore 0.4 of an inch, or $\frac{1}{373000}$ of the total length.

During the measurement of the base the party was caught in a heavy gale while returning from the island. One boat was nearly swamped, and the other, though she weathered the seas better, had still greater difficulty in getting back. The following morning all the native boys, except two, deserted.

AZIMUTH OBSERVATIONS.

An astronomical azimuth was observed with a 5-inch micrometer microscope theodolite at Weti, a small town on the western coast of Pemba, from which both ends of the base were visible. Two pairs of east and west stars were observed, and the value found had a probable error of 0.5″.

PRISMATIC COMPASSES.

All the prismatic compasses were tested by the observed azimuth and the magnetic variation of each noted. The compasses varied among themselves to the extent of 1° 50′. The firm of instrument makers who supplied them reported that, owing to the difficulty of making the central line of magnetization correspond with the long axis of the needle, variations of this amount were to be expected. It is therefore essential that the magnetic variation of each instrument should be determined.

The latitude of Weti was found from astronomical observations taken with a 5-inch theodolite. Four pairs of north and south stars were used. The latitude was found to be 5° 3′ 48″.3 with a probable error of 2″.9. This latitude agreed to within 1″.5 with the latitude as shown on the Admiralty chart. The latitude observations appeared to indicate that the refraction in the southern quarter of the sky was a good deal more than in the northern; the

pairs of stars which transited at the lower altitudes giving a greater value for the latitude than those which transited nearer the zenith.

Considerable difficulty was experienced in making the astronomical observations, owing to the sky being generally covered with clouds or haze.

TRIGONOMETRICAL TRIANGULATION.

Trigonometrical triangulation was only possible between the chain of islands and the west coast of the main island. Very few points could be fixed inland, owing to the lack of any prominent hills, and the dense vegetation. A large amount of clearing was required at each trigonometrical station, and on the average it took one day to establish each.

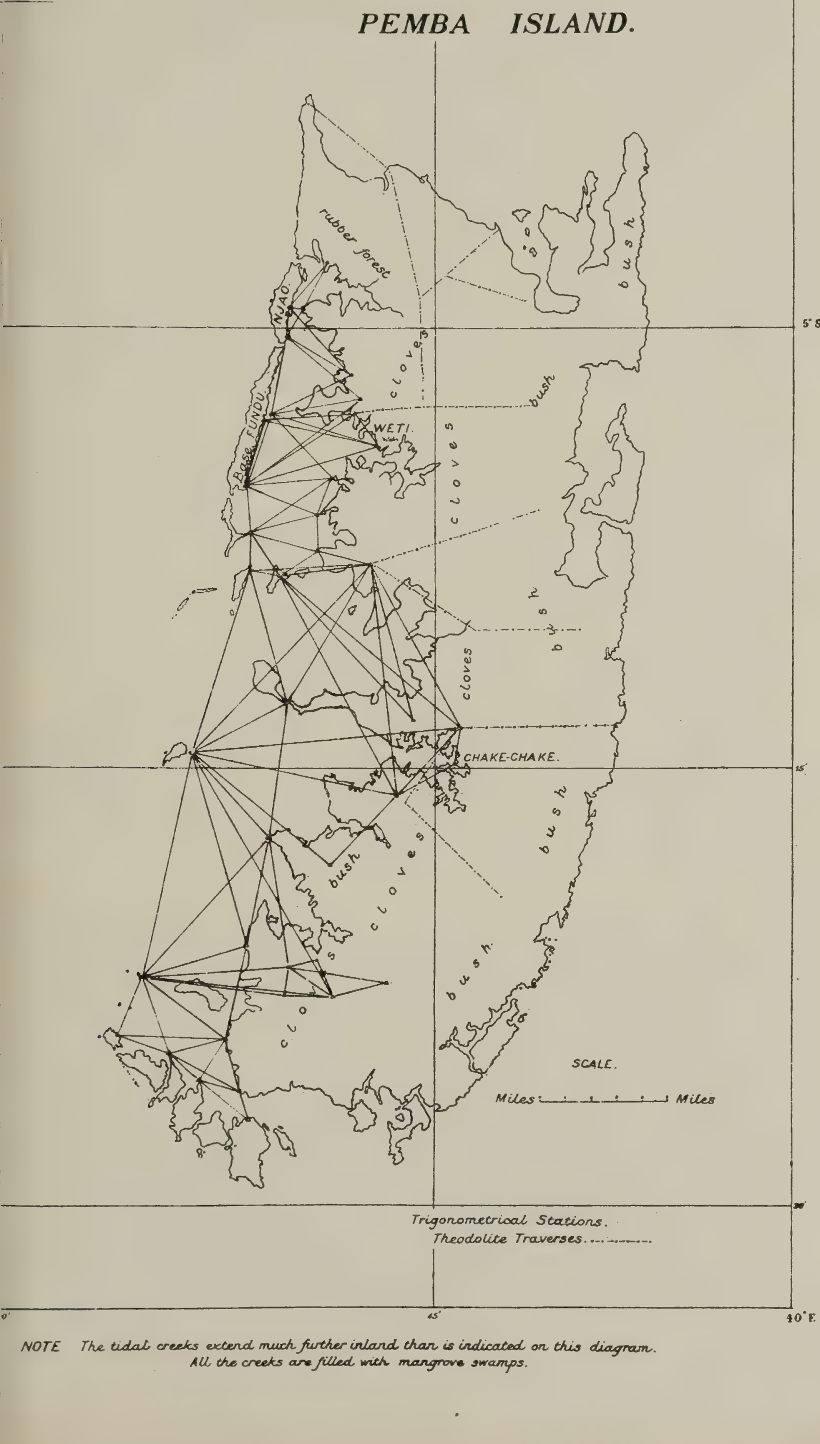
During the clearing operations a large tree that had been cut, but which was still held up by rubber vines, fell suddenly and broke Lance-Corporal Whitters' leg. A telegram was sent home to the Foreign Office asking for another topographer to take Lance-Corporal Whitters' place. Lance-Corporal McQueen was sent out, and arrived in Pemba about three months later. By that time Lance-Corporal Whitters had recovered from his accident, and returned to work.

Some of the trigonometrical stations were established in mangrove swamps and below high water mark, no other site being possible. At one important point a dhow sailed into the trigonometrical beacon at high tide and carried it away; the mast of the dhow also went by the board. Fortunately, all observations had been taken to and from this beacon before it was demolished.

The atmosphere was very thick, and great difficulty was found in seeing the trigonometrical beacons, especially when they were standing before a background of waving palms. Every form of beacon was tried, and the only one that proved to be visible was a straight pillar, a foot in diameter, and 10 to 15 feet in height. These pillars were made of light wooden poles padded with grass or brushwood to the required thickness. If the background was dark, the beacon was covered with white cotton cloth, but if the background was sea or sky, the beacon was left uncovered. A large flag was placed on the top of each beacon.

Many beacons were destroyed by the natives as soon as they were erected, and the materials stolen. Native police were told off to protect the beacons, but no arrests were made. Eventually, it was found that the best way of protecting the beacons was to slash the white cloth with a knife so as to render it useless for wearing purposes. An even more effective way would probably have been to splash the white cloth with red paint, which the natives would certainly have taken for blood. They regard a blood-stained cloth as "very bad medicine."

Compass bearings were taken from each new trigonometrical station to all stations visible from it, and noted in the angle books.



NOTE. The tidal creeks extend much further inland than is indicated on this diagram. All the creeks are filled with mangrove swamps.

Without this it was often found impossible for the observer to pick up the beacons with the theodolite telescope. The latter has such a small field that sweeping for beacons is a slow business, and the strain on the eyes of the observer is very great. If the compass bearing of the required beacon is known, no sweeping is necessary.

TREE STATIONS.

At three points tree stations were constructed. A large tree was cut some 20 feet from the ground, and the top of the stem then formed an observation pillar on which the theodolite stood. The legs of the instrument were not used. Round the stem, and some 4 feet below the top, a platform of sticks lashed with creepers was built for the observer.

It was found necessary to cut away all the branches from round the stem, otherwise the tree shook in the slightest wind. If a tree station more than 20 feet high is required, a very large tree must be selected, or the stem will sway a little to every movement of the observer. No one but the observer should be on the platform while observations are being taken. The observer should note whether any alteration of his position causes the cross hairs of the theodolite to move off the object. If this is so, he must observe each angle separately, taking care not to move his position during the observation. By using the reflecting eyepiece, and turning it as required, angles up to 180° can be observed without the observer moving his position, but to read both horizontal microscopes he will often have to unclamp the lower plate and swing the instrument round.

THEODOLITE TRAVERSES.

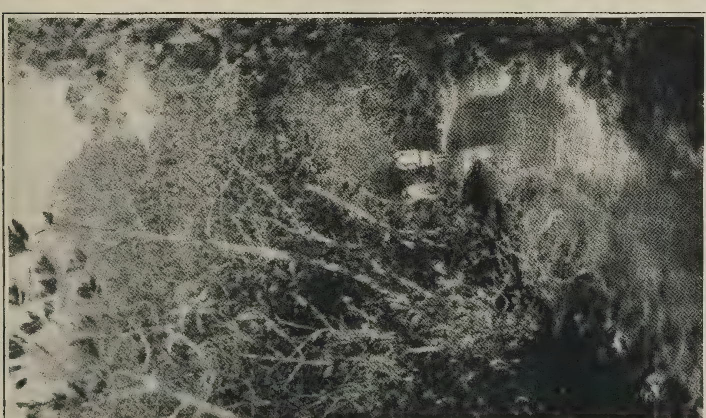
When the triangulation had been carried as far as possible, Lance-Corporal Cager began the topography of the western coast, working north from Chake-Chake, the capital of the island. Captain Craster and Lieutenant Kyngdon proceeded to run theodolite traverses across the island from trigonometrical stations on the western coast.

The traverses involved very heavy clearing, and as the clove harvest was just beginning, the labor was very limited. Consequently, progress was very slow. At first the traverse lines ran through the clove plantations, and here great care had to be exercised to select the lines that involved least cutting, as heavy compensation had to be paid for each tree cut. After the clove plantation had been left behind, the lines entered thick bush with only occasional patches of cultivation. In the thicker parts of the bush, felling the trees was not sufficient, as they lay piled up to a height of 6 or 8 feet in the traverse clearing. It was therefore necessary to cut up each tree and remove it piecemeal. This reduced the rate of progress to about 300 yards a day.

Over the trees grew in great luxuriance a creeper bearing a fruit like bunches of white grapes. The leaves of this creeper when crushed under foot gave out a strong aromatic scent that tainted

A bush party.

A main road through clove plantations.



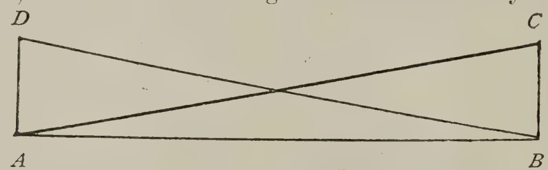
A path through a mangrove swamp.

the atmosphere in the whole neighborhood, affecting even the native boys with a feeling of suffocation and giddiness. Another creeper bore pods covered with minute "hairs," causing the pods to appear as if made of yellow plush. When the creeper was shaken, the air was filled with these tiny yellow "hairs," which penetrated the pores of the skin and caused intense itching and irritation.

On several occasions the traverse line had to be altered, to avoid disturbing the nests of wild bees. A professional honey gatherer accompanied the cutting party, to burn out and remove the nests, but when there were many nests close together even the professional could not tackle them.

It was not found possible to close the traverses on each other, as had been the original intention, owing to the heavy clearing that would have been required. The native boys also refused to do any more work in the dense bush belt that fringes the eastern coast of the island.

Each traverse point was marked with a permanent signal, so that the topographers were able to identify it. Both forward and back vertical angles were observed with the theodolite from each traverse station, and the vertical height was calculated by the formula



$h=c$ tan S; where S is half the algebraic sum of the angles observed from each end of the leg, and c is the length of the leg.

The magnetic bearing of each traverse leg was entered in the field book. This was found useful as a rough check on the reduced bearing of the leg as calculated from the theodolite observations. If the ground had not a uniform slope between two traverse stations, a vertical angle was observed with a clinometer at each point where the slope changed, so that the taped measurements could be reduced to their true horizontal lengths.

SUBTENSE MEASUREMENTS.

When possible, the length of the traverse legs was measured by subtense methods. This rendered the clearing of the line for tape measurements unnecessary, and reduced the amount of bush cutting to a minimum.

Only two signals were used in the subtense measurements. The first, distinguished by a flag, marked one end of the base and the traverse station; the second marked the other end of the base.

In the above figure A and B represent two traverse stations and AB the traverse leg. BC is the base measured at right angles to AB and C is the subtense signal. The angle BAC was measured with a theodolite, two sets of five readings each being taken on each face. The length of AB was then worked out from the formula AB=BC cot BAC.

Before leaving A the observer measured a base AD at right

angles to AB, and set up a signal at D. On reaching B, he observed the angle BAD, and found a second value for AB from the formula AB=cot DBA.

The mean value of AB was taken as the true value, but when the bases AD and BC differed greatly in length the value of AB as found from the longer base was given the greater weight.

Subtense methods were also used for fixing prominent features which could be seen from one trigonometrical station only, and which could not therefore be fixed by triangulation.

Six theodolite traverses were run across the island. Their total length was about 40 miles and there were 218 traverse stations. The labor involved in calculating the coordinates and heights of the latter was very heavy.

PROJECTION USED.

The trigonometrical and traverse stations were plotted on the map from rectangular coordinates. As the end of the island is small, no appreciable error was introduced by this method.

TOPOGRAPHY.

The island was surveyed on a scale of 1 inch to 1 mile.

Owing to the absence of prominent hills and the density of the vegetation, ordinary plane table methods could not be used, and the following procedure was therefore adopted. Prismatic compass traverses were run between trigonometrical and traverse stations, so as to cut the country up into triangles of 3 to 5 miles side. These traverses were plotted on foolscap paper on a scale of 6 inches to 1 mile. The closing error of each traverse was distributed among the traverse stations, and the corrected coordinates of each station measured. The traverse stations were then plotted on the plane tables from their corrected coordinates, and the detail was plotted from the traverse field books.

The topographers subsequently took the plane tables into the field and filled in all the remaining detail, and the contours, by plane table traverses run between the points already fixed. As these traverses were short, the closing errors were small, and could be adjusted in the field. The plane table was set by means of the trough compass.

As it was generally impossible to see more than 10 yards in any direction, a native boy was sent about 200 yards ahead to act as a point, and a compass bearing was taken—or, in the case of a plane table traverse, a ray drawn—on the sound of his voice. At 200 yards the probable error in the bearing was found to be about 2°. The error, however, increased very rapidly with the distance, and at 400 yards it was about 5°.

Great difficulty was found in locating the point, if high ground intervened between him and the observer. The point was therefore instructed to halt at the top of each hill. It was found that a low pitched note was much more easily located than a high pitched one, and for this reason a boy with a clear bass voice was always selected as point.

Contouring was very difficult, as no observations of any value can be taken with a clinometer on the sound of a voice. Fortunately, the valley bottoms, though often swampy and impassable, were fairly clear and their slopes very uniform, so that it was possible to determine their general gradient by a few clinometer observations. The topographers therefore mapped the main valleys as soon as possible and marked along them the heights as calculated from their general gradient, starting from the high tide mark. The heights of the hills were afterwards fixed by reference to points in the valleys. The determination of heights by aneroid readings was tried, but proved a complete failure owing to the atmospheric pressure being very variable.

The country offered many difficulties to the topographers. Steep razor-backed ridges, about 200 feet in height, alternated with deep crooked valleys. The bottoms of the valleys were marshy and often impassable, and tidal creeks filled with impenetrable mangrove swamps intersected the country, so that long detours were necessary. Owing to the nature of the work the topographers could only survey half a square mile a day. Much time was also lost from wet weather.

The slow progress made by the topographers and the urgent necessity of completing the work before the rainy season, made it essential for the two officers to undertake a considerable part of the topography. In addition to running many of the prismatic compass traverses for the topographers, they mapped some 70 square miles of the island.

As a rule, it is not desirable that officers should do topography, because it is work that can be done by non-commissioned officers and men who are, of course, on much lower rates of pay, and when an officer is employed as a topographer he can not exercise enough supervision over his party. But, owing to the heavy demands for trained topographers, foreign survey parties must generally be satisfied with the minimum rather than the most economical number.

The area of the island was originally estimated at 300 square miles, and the time allowed for the survey was eight months. An area of 300 square miles was surveyed within the time allowed, but the island proved to be about 400 square miles in extent, and the survey took nine and one-half months.

NAMES.

Great care was necessary to obtain the correct names of the various districts and villages, as the common pronunciation often gave no clue to the spelling. Many of the villages had opprobrious names, such as ''The Fools,'' ''Fly Blown,'' etc., which the inhabitants would not admit, but which were used with gusto by their neighbors. One little congregation of poverty-stricken huts was endowed with a name which meant ''Because I have nothing, my brothers won't visit me.'' Lists of all names that would appear on the map were made out and forwarded to the assistant col-

lector of the district who, after consultation with the headmen, wrote the correct form against each name.

LONGITUDE SIGNALS.

An attempt was made at the end of the survey to determine the longitude of Chake-Chake, the capital of the island, by wireless telegraphic signals from Zanzibar. The attempt failed, owing to continuous atmospheric discharges, due to thunderstorms over the mainland of Africa, which made themselves heard on the wireless receiving telephone, and which were indistinguishable from the time signals. It often seemed as if the wireless signals themselves provoked atmospheric discharges, for the latter often came as an echo to the former. Perhaps it is possible that a cloud which is already fully charged may receive from the wireless signal an additional charge which is sufficient to break down the insulation of the atmosphere and cause the cloud to discharge itself.

Even under the best conditions, the wireless telegraphic apparatus is not very suitable for longitude signals. It is almost impossible to send a short sharp signal, as the heavy currents used produce an arc at the contact points of the signalling key, and so prolong the signal for an appreciable interval after contact has been broken.

Chronometers installed in a wireless telegraph station are sure to be affected by the heavy currents and strong magnetic fields, and can not, therefore, be expected to keep a uniform rate.

INSTRUMENTS.

Two 5-inch micrometric microscope theodolites, made by Messrs. Troughton & Simms, were used during the survey. The sights on these instruments are rather crude, and at night it is difficult to align the telescope by them. In other respects the theodolites are very perfect instruments.

Four-inch prismatic compasses, graduated to 30′ and reading by estimate to 10′ were used. These were mounted on light tripod stands and proved very satisfactory. The Indian clinometers which were of standard pattern gave some trouble, because they contained so much steel that they deflected the compasses if in their vicinity. The plane tables were 24 by 18 inches in size and mounted on heavier and more rigid legs than are usually supplied.

Two measuring wheels (perambulators) were taken out, but could not be used, as the paths were too rough. The party was equipped with heliographs, but the cloudy weather prevented their use.

Great difficulty was found in preventing the steel measuring tapes from rusting. The only way to preserve them was to fill up the leather case with paraffin oil at the end of each day's work. It would be a great advantage in damp climates to have a small oil tank in which the tapes could be stored when not actually in use.

TOOLS.

Axes and saws were purchased in England. Large bush knives

were obtained locally in Zanzibar. When much clearing has to be done it is best to issue an axe and knife to each boy, and let him keep them until the work is finished. He then takes care of them, and keeps them well sharpened. For heavy bush clearing, about one-third of the party should be equipped with felling axes and two-thirds with hand axes. Axes to be used by natives should be of the lightest English pattern.

Much trouble was experienced in fitting new helves to the axes, as the sockets in the steel heads were very small, and the native wood helves had to be whittled away to fit the sockets. The new helves in consequence broke off at the socket after a few days' work. Spare ash helves were taken out, but these became dry and rotten before they were required for use.

Large cross-cut saws with teeth set very wide proved extremely useful, when once the native boys had learnt to handle them.

Many of the boys carried sheath-knives with blades about 6 inches long. For cutting creepers and small brushwood these were much more handy than the large bush knives, and it would have paid to have equipped the whole party with them in the first instance.

CAMPING ARRANGEMENTS.

Each officer and non-commissioned officer was equipped with a double fly ridge tent, camp furniture, cooking utensils, and a filter, so that he could camp alone if necessary. In addition there were two mosquito-proof houses; one for the officers and one for the non-commissioned officers. The houses consisted of a mosquito curtain 8 by 8 by 8 feet in dimensions. A small sheet, 12 by 12 by 8 feet, thatched with palm leaves, was built at each camping ground, and the mosquito net hung from the rafters. The windward sides were filled in with palm leaves to keep out the driving rain, but the leeward sides were left open. Without these mosquito-proof houses it would have been impossible to do any work after sunset, and there is no doubt that they saved the party from a great deal of fever.

Encouraged by the example of the survey party, one of the Zanzibar Government officials employed by the agricultural department to supervise the gathering of the clove crop camped in the interior of the island for some months, but did not use a mosquito-proof house. He suffered from repeated attacks of fever, and died in Zanzibar a few days after leaving Pemba. With the exception of the survey party, no other European has ever attempted to camp for more than a few nights in the interior.

Owing to the very heavy dews and frequent rain, the native boys and servants could not bivouac as they do in most parts of Africa. Native huts were therefore hired for them, and when these were not available thatched huts were built.

HEALTH.

Five grains of quinine a day were taken by every member of the

party as a preventive, but all suffered from malaria. There were no cases of blackwater fever, though it is very prevalent in the island, and often follows an attack of malaria if the patient returns to work too soon. The doctor therefore gave instructions that no malaria patient should return to work until four or five days after the fever had left him.

Two of the party suffered from severe congestion of the liver, accompanied by high fever, as the result of chills. It has been suggested by a doctor of considerable African experience that these attacks may have been due to the bite of a tick, which produces similar symptoms in animals. This is a disease that only a doctor can diagnose, and if the proper remedies are not applied early abscess of the liver is likely to follow.

Owing to the necessity for finishing the survey before the rainy season, it was necessary to work ten hours a day for seven days a week, and at the end the whole party were much exhausted by the hard work and the effects of malaria. In fact, ten months may be taken as the maximum time that a European surveyor can work at a stretch in the unhealthier parts of Africa. At the the end of that time he should have at least two months' rest in a bracing climate before he returns to work.

The completion of the survey before the rainy season was due entirely to the zeal and energy displayed by Lieutenant Kyngdon and the three non-commissioned officers. Had the rains broken before the outdoor work was finished, the survey would have been prolonged for another two or three months, and the cost would have been increased by not less than 20 per cent.

River and Harbor Notes from Foreign Lands

REINFORCED CONCRETE LOCK AND DAM ON THE KÖRÖS AT BÖKÉNY, HUNGARY.*

* * * * * * *

When the Minister of Agriculture of Hungary decided upon canalizing the Körös, and building a dam and lock at the mouth of that river at Bökény, the Department of Water Supply suggested that reinforced concrete should be used, owing to its light weight, not only for the construction of the dam, but also for that of the lock.

The works were completed in 1906. The plans were drawn up by Mr. Constantin Zielinski, a civil engineer and professor at the Engineering Polytechnic School of Budapest.

His plans provided for the application of reinforced concrete on a large scale, not only in the side walls and the invert but also in the foundations.

DAM.

The length of the dam of the Poirée type is 35 m. and the width, measured in the direction of the current, is 21.35 m. The height of the lift above sill level is 3.2 m. and the total difference in level is 3 m. The dam invert is protected against undermining at the toe by a sheet-piling retaining wall 52 m. in length, 4.5 m. high, constructed of reinforced concrete piles. These piles, shown in Fig. 1, were driven to the necessary depth partly by a water jet, partly by a monkey, and partly by these two methods used jointly. In order to render the work quite water-tight, a circular hole was provided between each two piles. The loose material in this space was afterwards scooped out by a water jet, and the gap grouted with cement mortar.

The invert of the dam is founded on piles of reinforced concrete. The reinforced concrete beams constitute the main framework of the invert and are borne by these piles, and their reinforcement is connected with the reinforcement of the latter. The invert itself is made of slabs from 0.10 to 0.12 m. thick, which are built in between the beams. The empty squares formed by the beams and secondary beams are then filled with poor concrete in mass.

All the piles which carry the invert (see Fig. 2) are built in the same manner but are used for different purposes, according to their position. Whilst those above the sill and those below the pair of

*From a Report by the Hungarian State Water Survey, Proceedings Permanent International Association of Navigation Congresses, Philadelphia, 1912.

pivots in the lower end of the work are only required to resist pressure in one direction, those below the sill are probably subjected to an upward pressure of water produced by percolation of water from the upper reach. In order, therefore, that this upward pressure should not lift the invert, the latter is fastened down to the piles, and the piles themselves are provided with side grooves.

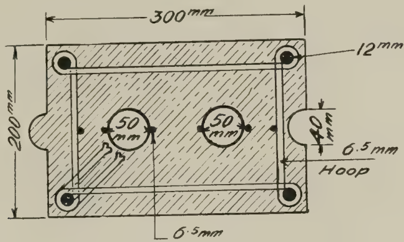


Fig 1.

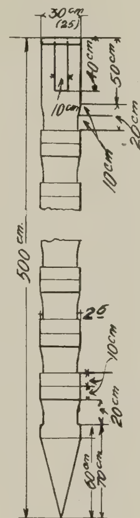


Fig. 2.

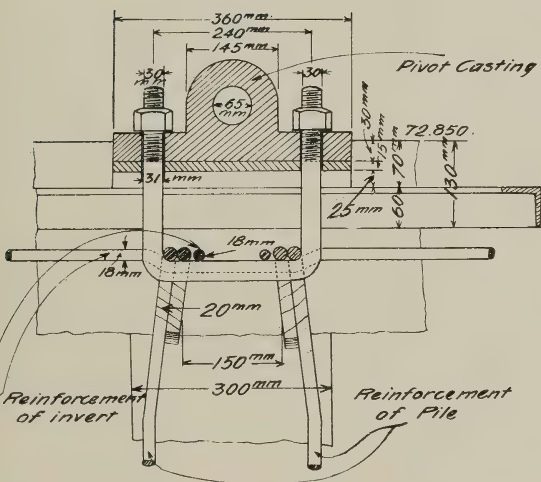


Fig. 3.

The same considerations influenced the design of the piles which occur underneath the pair of pivots in the upper end (see Fig. 3). Tests which have been carried out show that the force required to shift the driven piles is considerably in excess of the compression and tensile forces which they have to resist.

The piles under the invert were driven mainly with the aid of a water jet. Two pipes were used to convey the water under pressure to the shoe of the pile (see Fig. 2). This method succeeded admirably in pure sand, and the pile was driven to a depth of 4.5 m. by means of water under a pressure of 2 to 2½ atmospheres in

the space of four to five minutes, not counting the time taken in placing the pile in position prior to sinking it. Whilst the piles were being sunk with the aid of the jet, the water scooped the soil around the piles, leaving large cavities, so that after the piles were completely sunk they had to be held in position for a few moments. No trouble was experienced in doing this, and in the course of five to ten minutes the sand filled up the cavities, so that the temporary supports could be removed. In half an hour or so the molecular forces of the subsoil had reached such a degree of stability that it was impossible to cause any movement in the piles by hand. In the clay portions of the soil, the invert piles were driven by blows, and in some instances were driven in combination with a water jet, just as in the case of the reinforced concrete sheet-piling.

* * * * * * *

Regarding this dam, we may mention that before work on the reinforced concrete invert proper was commenced, a layer of mass concrete from 0.20 to 0.25 m. thick was deposited at the bottom of the excavation, so as to form a suitable level foundation for the reinforced concrete part of the work.

LOCK WITH A LOCK CHAMBER.

The available length of the lock is 70 m. and its width, between the ends of the lock and also between the side walls, is 10 m. The maximum lift is 3 m.

The lock is not built along the river, but along a lateral canal. The soil was more favorable there than in the case of the dam, and for this reason the lock was not built on piles, but the invert was made stronger instead.

As in the case of the dam, reinforced concrete sheet-piling is used to protect the lock against scouring. We also find that a layer of mass concrete 0.20 m. to 0.25 m. thick has been used in this structure to level the foundation, as in the case of the dam described above.

The invert consists of T-shaped reinforced concrete beams spaced 1.36 m. apart, connected together. The depth of the beams in the lock chamber is 0.70 m. The square gaps between the reinforced concrete beams and secondary beams are filled with poor concrete in mass.

The invert has been calculated as a girder, resting on an elastic foundation, and exposed to a uniformly distributed load throughout its length.

The loads are represented by the weights of the side walls, and the embankments, and by the lateral thrust of the earth. The data used for several locks built in Hungary on similar kind of soil has been taken as a basis in the calculations of the elasticity and compressibility of the soil. This data has been obtained by measuring the settlement from time to time, due to the increasing loads as the lock was built. It has been assumed also that the reactions of the soil which balance the weight of the invert are

in direct proportion at each point of the invert to the actual settlement at that point. Taking this as a basis of calculation, the following dimensions have been obtained for the invert beam: Depth, 0.70 m.; breadth, 0.25 m.; width of slab, 0.12 m. The reinforcement consists of two round 35 mm. rods, and one round 40 mm. rod.

The side walls of the lock are ribbed, similarly to the retaining walls. These ribs are connected to the invert beams, so that the whole of the lock forms a monolithic structure. The ribs are placed so near together that the slab between them need only be 0.10 m. thick. It was only in the lower part of the side wall which

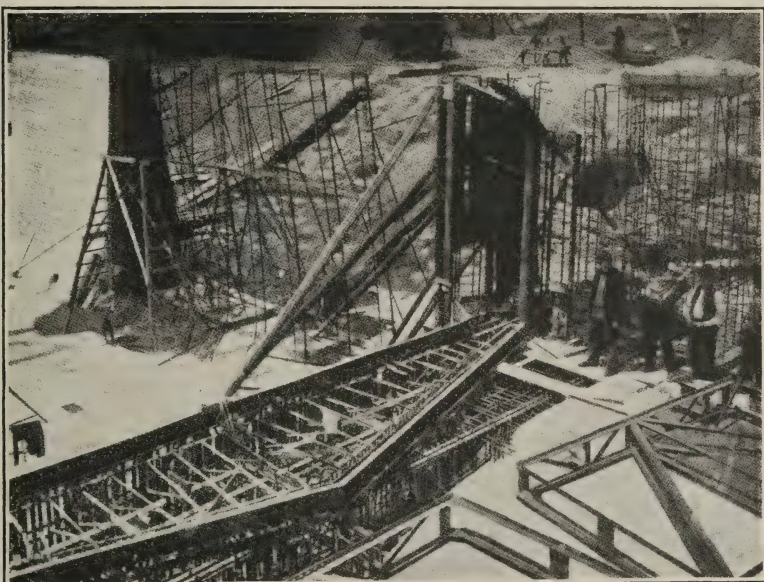


Fig. 4.

holds the filling culvert which passes through the ribs, that larger dimensions and stronger reinforcement had to be adopted.

No dressed stone has been employed in this lock, but the exposed portions, such as the hollow quoins and the sills, are protected instead by steel flats or angle irons. Fig. 4 shows this method of construction.

* * * * * * *

The side walls are not faced, and for this reason they are protected against the blows of craft by a number of vertical timber balks. These balks are naturally connected to the ribs, so that the blows from the vessels may be transmitted directly to the latter.

Fig. 5 shows the ribbed side walls during construction and a portion constructed.

We may mention here that in both the dam and the reinforced concrete lock, water-tight inverts 10 m. and 18 m. long have been constructed, one above the work and one below it. The inverts and the revetments of the banks which are borne by them are made of water-tight reinforced concrete slabs, with the object of leading the flow of the water from the upper pond and protecting still further the dam and the lock against scouring. The slabs on the revetment of the banks are 1 square meter in area; they are molded beforehand, and are designed to interlock with one another. They were laid side by side on the bank, and anchored at their corners into the soil by a small pile of reinforced concrete; the small gaps remaining between the slabs were afterwards filled in with concrete. The revetments of the inverts above and below the work were made in situ of slabs 4 square meters in area, which were likewise fastened down at the corners.

From the preceding we see that the dam and lock of Bökény consist entirely of reinforced concrete, and for this very reason we are able to deduce conclusions from this experience which may be applied to the employment of reinforced concrete for other similar works.

In this connection, we have hardly anything to say respecting the dam, except perhaps to repeat that the construction of the invert in the manner already described—that is to say, the anchoring to piles, the construction of the sill, and the methods of fixing the pivots—has been a complete success.

We shall confine our remarks preferably to the lock, and we shall not only call attention to its advantages, but also to its disadvantages.

In the first place, we should point out that in hydraulic works which are built on the alluvial sand of the large Hungarian plains, the most important point to consider is the type of foundation to adopt, which is also intimately connected with the question of excavation.

Construction of foundations by compressed air would certainly be the best method to employ, but it is hardly suitable for this work on account of its high cost, and we are therefore compelled to carry out this construction in open trench. One can not readily remove the water which flows into the open trenches without disturbing the soil. In general, one can not lower the level of the subterranean waters by sinking pits near the excavation to collect the waters, because the sand on the Hungarian plains is so fine that the water will only fill these pits to a slight extent. The only way which remains is to concrete the inverts under water, or else to divert the subterranean springs into collecting sumps, and pump the water out of these. The excavation of the lowest layers is more often than not a very onerous task. Experience shows us that the simplest and cheapest method to adopt, before the re-

moval of the final bottom layer of 3 or 4 meters, is to surround the inverts to be constructed by water-tight sheet-piling, and then to excavate this earth when thus protected against the inflow of water.

Generally speaking, water is tapped from the springs during the execution of the work. These springs, which generally destroy the foundation, are found in greater frequency and intensity as the depth of the excavation is greater.

These considerations lead us briefly to the conclusion that in such circumstances the inverts should be built in reinforced concrete. The reasons for this are as follows:

1. A reinforced concrete invert is very much thinner than a mass

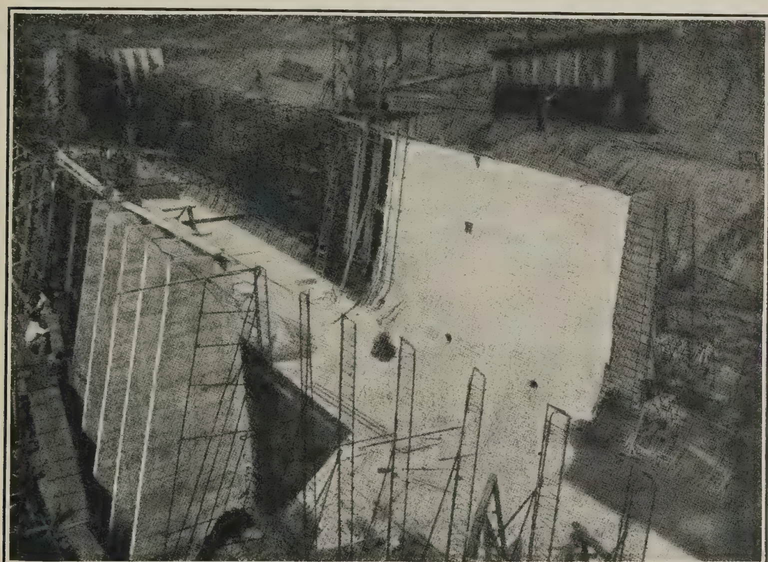


Fig. 5.

concrete invert. For instance, the invert of the Bökény lock, including the mass concrete foundation, is only 1 m. thick, whereas the inverts of our other locks which have been founded on the same kind of natural soil are from 2 to 3 m. thick. Consequently, the adoption of reinforced concrete enables one to dispense with that portion of the work which is the most difficult and costly, namely, the excavation.

2. As the excavation need not be carried down so deep one can either dispense with the sheet-piling or at least reduce its dimensions.

3. The foundation soil is disturbed to a lesser degree, because the springs are the less frequent, and the less intense the shallower the foundation.

4. Reinforced concrete inverts are much less rigid and can take much more unequal loadings than an invert of mass concrete which is two or three times as thick. For instance, the invert in the Bökény lock settled down nearly 10 mm. more near the side walls than along the center of the lock without cracking. The dangers which arise from unequal settlement are minimized when the invert is built of reinforced concrete, without counting that even if the concrete should split the reinforcement ensures a temporary protection.

It goes without saying that inverts can only be satisfactorily constructed in reinforced concrete, provided the excavation or trench is quite dry. This applies particularly to the case of an invert with complicated reinforcement, like the one at Bökény. In this instance the conditions were fairly favorable for this lock, as it was built in a short cut connecting a bend of the river. The excavation was consequently 200 m. away from the bed of the Körös.

The upper and lower bends of the cut were only excavated after the lock was built. Furthermore, during the construction of the invert the water level of the river happened to be favorably low, so that a layer of mass concrete 0.30 m. thick was sufficient protection against the spurting of water from springs. This dimension under less favorable conditions would have to be considerably increased.

It is a remarkable fact that the designer has kept strictly in his design for the invert of this lock to the characteristic form of reinforced concrete structures in which beams and slabs alternate (ribbed slabs). This example is the practical expression of the designer's particular views on the subject. We do not consider, however, that this method is a good one to follow. We are of opinion, on the contrary, that the reinforcement of the invert should be as simple as possible, and thereby easier to place, owing to the obstacles which are found in excavations and hamper the work of placing complicated reinforcement.

We have already said that the lock is a monolithic structure. We have pointed out the disadvantages of monolithic structures of this type; owing to the heterogeneous nature of the foundation soil, the settlement of the upper end of the lock was greater than that of the lock chamber and of the lower end of the lock, which caused a number of thin cracks on the top 3 or 4 meters of the side walls, and also a much bigger crack in each side wall. The reason being that the upper part of the side walls at the height of 6.5 m. was not able to adjust itself to this unequal settlement. We must say, however, that we only found cracks in the upper part of the side walls, and that the lower part as well as the invert were free from cracks, although the invert was only slightly reinforced in the longitudinal direction. This fact confirms the opinions of those who hold that it is necessary to provide expansion joints in locks, not only in the invert, but also in the side walls.

We must also call attention to the fact that the side walls of the Bökény lock are not faced. It is true that the exposed concrete will weather fairly well, as the concrete employed has been carefully selected and the work carefully carried out, but we should consider for future work whether it would not be desirable to face the concrete with stone or some other suitable material, as concrete tends to chip away under frequent shocks.

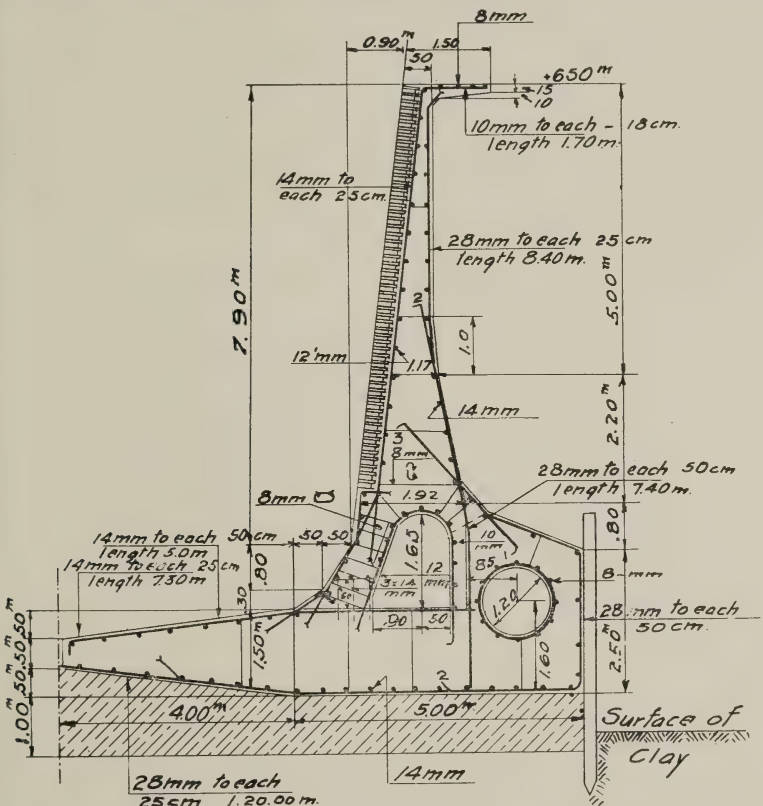


Fig. 6. Cross section of wall of reinforced concrete lock near Budapest.

REINFORCED CONCRETE LOCK ON SMALL BRANCH OF DANUBE, NEAR BUDAPEST

To resume briefly, these were the remarks which we thought fit to make in connection with the dam and lock at Bökény, and which we have endeavored to apply profitably in the case of a lock now in course of construction in one of the branches of the Danube, near Budapest. We propose to say a few words concerning this lock.

The conditions of the foundations are quite different in this lock to what they were at Bökény. The foundation is very stiff clay,

which belongs to the upper oligocene layer with a permeable layer of large gravel underneath it. In this case it was essential to provide perfect drainage.

The Hungarian State Water Survey organized a competition for plans for part of the lock. A few competitors sent in projects of reinforced concrete, of which one is shown in Fig. 6. The author of this scheme proposed to surround the excavation by wooden sheet-piling and then to place a layer of mass concrete from 1 to 1.5 m. thick between the piling, as shown in the illustration. The object of this layer is partly to prevent the water from springing up from the soil, and partly to distribute the weight of the lock over the hard clay. The author proposed then to rest the lock itself upon this layer of concrete, or, in other words, to build two self-supporting retaining walls. A longitudinal joint was provided along the center of the lock in the direction of its

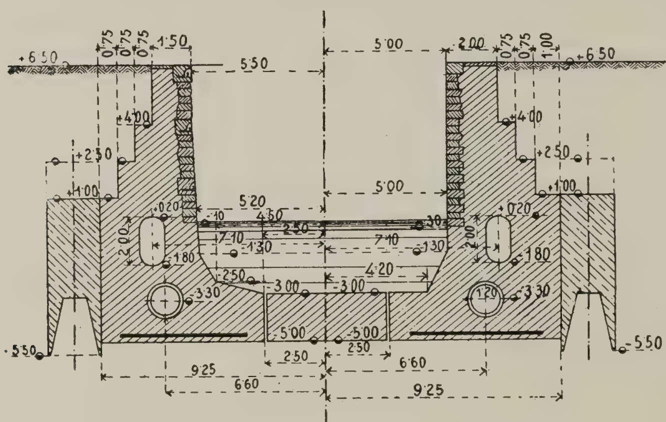


Fig. 7.

length, and this would have been grouted with asphalt or cement mortar, after the complete settlement of the lock. In addition to this, cross joints would have been provided so as to divide the side walls as well as the invert into separate sections which could settle independently of one another. The walls would have been faced with hard fire-bricks.

Several competitors attached the greatest importance to rendering the work as water-tight as possible. One of them proposed to surround the excavation by caissons sunk under compressed air. The caissons are very simple in construction; they are of concrete, their framework is of timber, and they are also slightly reinforced with steel.

As this design was the best, both from the point of view of cost as well as strength of foundations, its author was entrusted with the work. The Department of the National Water Supply acquired several of the other schemes and adopted some of their

suggestions in the final design of the lock. The cross sections of the lock are shown in Figs. 7 and 8.

The following points were considered in preparing the design:

1. To eliminate as far as possible the consequences of unequal settlement.

2. To employ reinforced concrete only at those parts where its use is justified by the want of space, or by heavy loads.

3. To protect the side walls and the heads of the lock by facings.

In order to meet these conditions the cross section of the lock was divided into three parts (Fig. 7), namely, two self-supporting retaining walls, and an intermediate invert. The toe of the side walls has steel reinforcement to provide against shearing stresses in the concrete.

The joints between the separate parts will be closed by corrugated copper steels, as shown in Fig. 9. The cross joints will be rendered water-tight in the same manner.

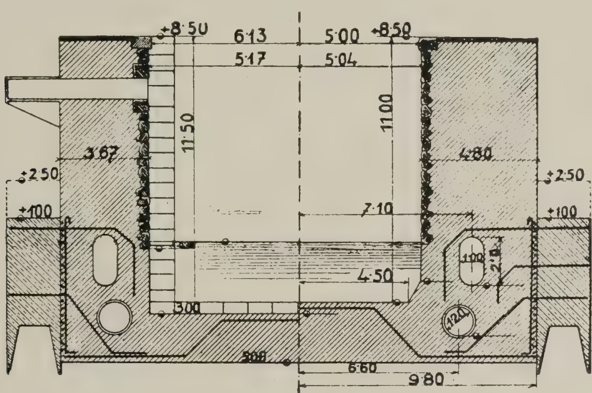


Fig. 8.

No provision has been made for a waterproof layer under the lock, as we anticipate that the caissons which will be sunk right into the clay will be sufficiently water-tight.

The side walls will be faced with hard limestone, which will resist the frost and reduce the cost of upkeep.

A continuous invert will be adopted at the heads of the lock, with reinforcement as shown in Fig. 8.

The walls will be connected by horizontal rods with the surrounding caissons.

In concluding our paper we desire to lay stress once more upon our opinion, based upon our experience, that the use of reinforced concrete is justifiable in the construction of locks with a lock chamber. But we must not allow ourselves to be carried to the other extreme. We must not build all the parts of the lock with reinforced concrete which has complicated reinforcement, as we

consider that the proper material to employ for such portions is not reinforced concrete, but concrete in which angle and channel irons are embedded.

There are, however, some soils where reinforced concrete proper is able to compete with success with any other form of construction. We mean, for instance, in cases where the locks have to be built on a soil which has little resistance. The side walls, as a matter of fact, need only then be considered and calculated as retaining walls, whereas if we had to build them as solid walls they would be too heavy. The stress on the foundations increases in proportion to the weight of the side walls. A side wall of double the weight produces twice the stress. If any slight undermining oc-

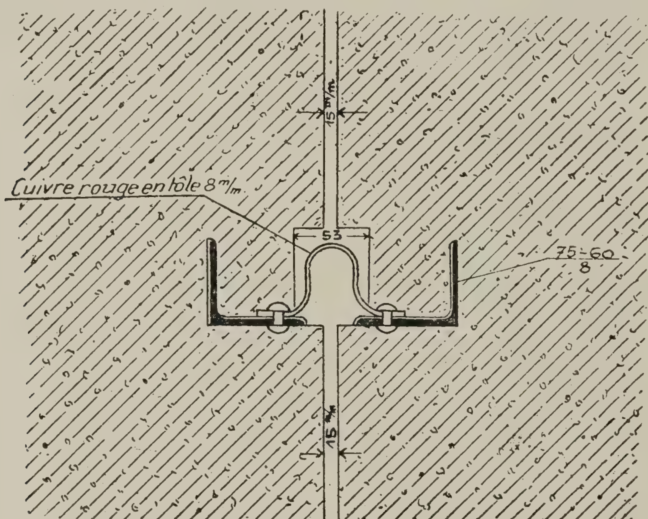


Fig. 9.

curs anywhere under the foundations the carrying capacity of the ground is still further reduced, and we can see at once that heavy side walls will increase the tendency to unequal settlement, and some very dangerous shearing stresses, leading to undermining, will occur.

In such circumstances, the proper course would be to build the side walls of a light section in reinforced concrete, and also, if necessary, to hold down the inverts by means of piles driven into the ground; similarly, for instance, to the method adopted in the case of the Bökény dam.

REINFORCED CONCRETE LOCK ON TOURA-TOBEL RIVER, WEST SIBERIA,* AND AT RYBINSKI, ON THE VOLGA

In all countries the adoption of reinforced concrete for hydraulic works has occurred very much later than is the case with any other method of engineering construction. This is due to the peculiar conditions which are inherent to this class of structures, which are of such large dimensions as not to enable all the advantages of reinforced concrete to be utilized. As these conditions also exist in Russia, the same delay in the application of reinforced concrete to navigable waterways has occurred, and has even been aggravated by the cheapness of structural timber. For this reason we are compelled to confine our report to a small number of examples of this kind without deducing any general principles regarding the use of reinforced concrete. We will refer specially to the lock on the Toura-Tobel navigable waterway, to the retaining wall of the type specified for navigable waterways in general, to the lock of the fluvial port of Rybinski, and also to the irrigation works in Turkestan.

We will first describe the reinforced concrete work for the Toura-Tobel rivers, Western Siberia, on the waterway which is to connect the Tobel and Kama rivers through the Oural mountain range. As shown in Fig. 10, the lock chambers of this lock are of the same type as the lock on the river Harmas-Körös in Hungary, which is a particularly large lock. The lock chamber is 17 m. wide, in order to take the largest steamers which ply on the river Obe, and it is 275 m. long, so as to accommodate a tug and its train of barges. The depth over the sills is 2.3 m. in accordance with the standard draught of vessels adopted on part of the Russian waterway system. The drop at the lock is 3.20 m.

The invert of the lock, similar to the one in the lock of the Harmas-Körös River, is a concrete monolith 1 m. thick, strengthened by reinforced concrete beams 30 cm. wide, spaced 1.6 m. apart and connected together by a reinforced concrete slab 25 cm. thick, which occurs sometimes on the top of the invert and sometimes at the bottom, according to whether the bending moment is positive or negative. (Section EF, Fig. 10.)

The reinforcement of these beams consist of fourteen 26 mm. round rods in the part in tension and of seven rods of the same size in the part in compression. The invert beams are prolonged at each extremity under the walls of the lock, as shown on the illustration. This provides a continuous foundation common to the invert and the lock walls. The spaces between the reinforced concrete beams are filled with poor concrete in mass.

The lock walls are also built of reinforced concrete and consist of a thin outer vertical slab of varying thickness, namely 20 cm. from the base up to the water level of the lower reach, and 12 cm.

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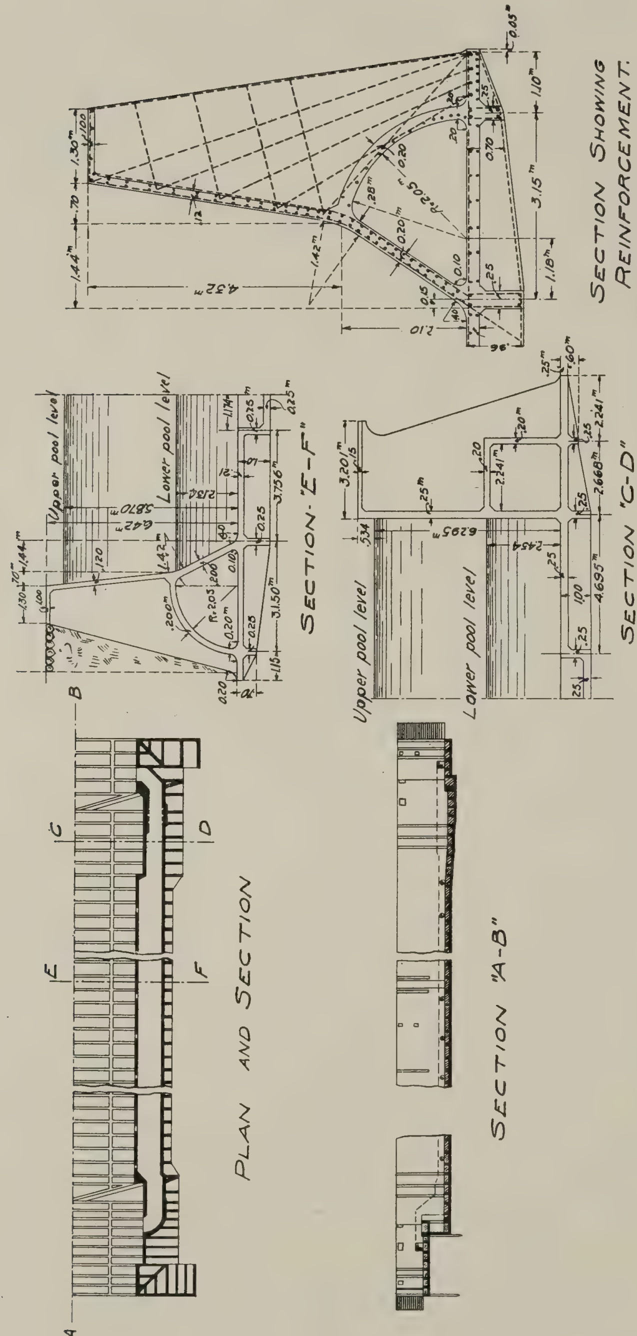
*From a Report by A. Nikolsky, Engineer in Chief of Ways and Communications, Schlüsselburg.



LOCK IN RIVER TOURA-TOBEL, WESTERN SIBERIA.

Fig. 10.

above that point. This vertical slab is supported by buttresses which bear on the invert beams. The outer face of the lock walls have a vertical batter of 1 in 2 in the portion between the invert and the water level of the lower reach and a batter of 1 in 10 in the part above. The top of the walls is 0.50 m. above the water level of the upper reach. It has a coping 1.25 m. wide and 10 cm. thick and the paving around the lock is brought up close to this coping.

A filling and emptying culvert is provided through the base of the lock walls in the form of a semi-circular arch (Fig. 10) which commences at the water level of the lower reach, so that the culvert is 2 m. high and 3 m. wide, and the reinforced concrete arch 18 cm. thick. The water from the culvert passes into the lock through a series of circular sluices spaced 5.40 m. apart.

The culvert is rectangular in section with walls 0.20 m. thick in the portions near the end of the lock. (Section CD, Fig. 10.) This shape of culvert facilitates the construction of the sluices in the culvert itself and enables a cofferdam of small beams to be built so that the lock chamber and the culvert can be completely emptied, if necessary.

We may remark that the whole of the height of the lower reach has been utilized in the construction of the culvert, so that the lock can be filled in the shortest possible space of time.

The arrangement of the reinforcement in the lock walls and buttresses is the same as that generally adopted in similar cases for taking the necessary stresses, and is shown in Fig. 10.

In calculating the reinforced concrete portions of the lock, it was assumed that the concrete would only take compressive stresses whilst the steel would work both in tension and compression. The coefficient of elasticity is assumed to be constant, and the ratio N of the ordinary coefficient of elasticity of steel to that of the concrete was assumed to be 15. The resistance of concrete to compression was assumed at 30 kg. per sq. cm. and its resistance to shear at ½ kg. per sq. cm. The tension in the steel was taken at 1,000 kg. per sq. cm. and the shear at 750 kg. per sq. cm. The adhesion between the steel and the concrete was taken at 4.5 kg. per sq. m.

Reinforced concrete was used in the lock of the above scheme so as to economize in the cost of construction as compared with an ordinary lock with masonry or mass concrete walls. This economy was realized as the cost of construction of a reinforced concrete lock was estimated at 3,750,000 francs, whereas this cost would have worked out at 4,500,000 francs for the ordinary type of construction, thus showing a saving of 20 per cent.

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Another structure which deserves attention is the lock now being built at the fluvial port of Rybinski, on the Volga. The size of the lock chamber, 150 by 12.80 m., enables it to accommodate the largest craft which ply along the river. For economical reasons, masonry is only used at the ends of the lock, and the lock chamber

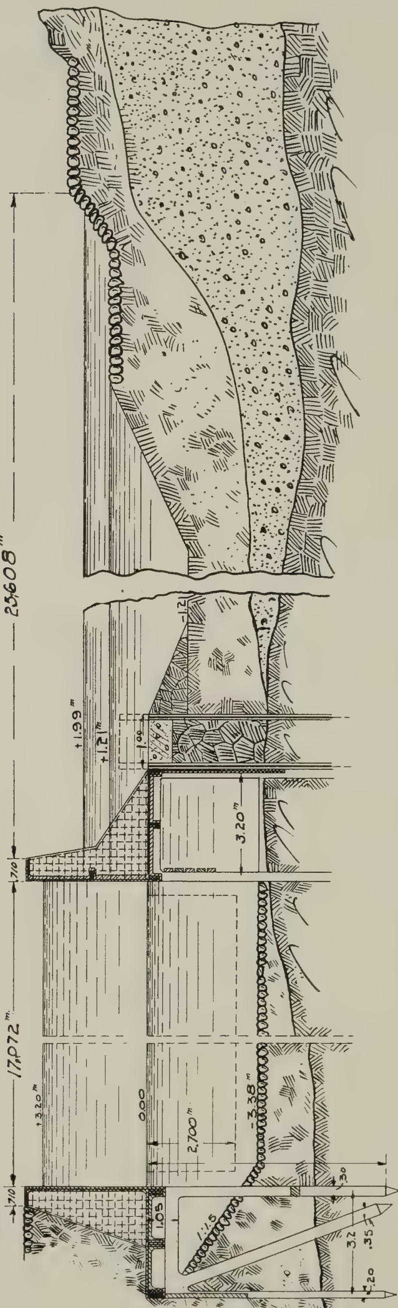

Fig. 11. Cross section of reinforced concrete lock at Rybinski, on River Volga, showing saving basin.

and invert consist of the earth merely covered by ordinary paving. The walls of the lock chamber are built in two different ways. On the side of the retaining pond, the lock chamber wall consists of two rows of piles and sheet piling 2.80 m. apart, with a top layer 71 cm. thick of mass concrete between the rows (Fig. 11). On the opposite side of the lock the wall is merely an ordinary bank with a slope of 3.2, paved over. This method of construction for the wall of the lock chamber is adopted for the portions between the invert and the low water level of the Volga. Above this level, and within the limits of the variations of the water level in the lock, the walls of the lock chamber are vertical, and of reinforced concrete, so as to economize the water in locking. These walls rest on reinforced concrete piles, which are spaced 3.20 m. apart along the length of the lock, and 2.13 m. apart in the direction at right angles to this.

In addition to the vertical piles on the embankment side, inclined piles are provided to resist the thrust. The piles in the front row are 30 by 30 cm. in section and those in the second row are 20 by 20 cm. The inclined piles are 35 by 35 cm. in section.

* * * * * * *

Sylvanus Thayer

Sylvanus Thayer (see frontispiece) was born at Braintree, Mass., June 9, 1785. He received a classical education at Dartmouth College, where he graduated, and was afterwards appointed cadet at the United States Military Academy, March 20, 1807. He graduated February 23, 1808, and was promoted Second Lieutenant, Corps of Engineers. During the remainder of 1808, he was occupied in surveying the sites and planning the batteries at Stonington and New Haven harbors, Conn., and was Inspector on the Fortification Works at Fort Trumbull, Conn.

During 1809, he served as Assistant Engineer in the construction of the defenses on the Massachusetts Coast, and from 1809 to 1811 was on duty at the Military Academy. Thereafter, until the outbreak of the War of 1812, he was Assistant Ordnance Officer in New York City. He was promoted First Lieutenant July 1, 1812, and served during the War of 1812 as Chief Engineer to Major-General Dearborn on the Niagara frontier in 1812; Chief Engineer of the right division of the Northern Army under General Hampton on Lake Champlain in 1813; Brigade-Major for Gen. Moses Porter's command in the defense of Norfolk, Va., in 1814-1815.

He was promoted Captain, Corps of Engineers, October 13, 1815, and Brevet Major, February 20, 1815, "for distinguished and meritorious services." In 1815 he was sent abroad for professional duty in Europe, examining fortifications and military schools and establishments, and had the opportunity of witnessing the operations of the allied armies before Paris, 1815, and of studying the military works and schools of France and the Netherlands, and of examining many famous battlefields and theatres of war.

In after life there was probably no other living man, save perhaps Jomini, so thoroughly and minutely acquainted with the campaigns of Napoleon.

He returned to the United States in 1817 and was appointed Superintendent of the Military Academy, in which position he served for practically sixteen years, during which time that institution was organized upon its present basis, and under his admini-

stration it became one of the most thorough, comprehensive, and successful military institutions of the world.

He was promoted Brevet Lieutenant-Colonel, March 3, 1823, "for distinguished and meritorious services," and Major, Corps of Engineers, May 24, 1828.

From 1833 until 1843, he was Superintending Engineer of the construction of Forts Warren and Independence in Boston Harbor, and continued in general charge of these works while on professional duty in Europe until 1846. During part of this period he was also in general supervision of the harbor improvements of Maine and Massachusetts and of the coast defenses of Boston.

He was promoted Lieutenant-Colonel, Corps of Engineers, July 7, 1838. From 1846 to 1857, he was Superintending Engineer of the construction of Forts Warren, Independence, and Winthrop, and of the sea walls in Boston Harbor.

In addition to his other duties, from 1833 to 1857 he was a member of the Board of Engineers for Coast Defenses and was President of that Board for more than nineteen years.

From December 21, 1857, to December 22, 1858, he was in command of the Corps of Engineers, exercising the functions of Chief of Engineers of the Army. Consistent with long-maintained views on this point, he declined to transfer his headquarters to Washington and, on his own application, was placed on sick leave of absence and so remained until June 1, 1863.

He was promoted Colonel, Corps of Engineers, March 3, 1863, and Brevet Brigadier-General, U. S. Army, May 31, 1863, "for long and faithful service."

On June 1, 1863, he was retired from active service "for having been borne on the Army Register more than 45 years." Previous to his death, he founded the Thayer School of Civil Engineering at Dartmouth College, New Hampshire, and established a public library in his native town of Braintree; and upon his death bequeathed about $260,000 for a free school in Braintree, limited to scholars "who shall have been born in Quincy, Braintree or Randolph."

He died September 7, 1872, and the Secretary of War, in announcing his death in orders, said, "The great worth and services of this veteran soldier are gratefully remembered by the graduates of the Military Academy, of which he is justly styled 'The Father,'

and his name will be entwined with the laurels which many of them have gained on the battlefield.''

The degree of A. M. was conferred upon him by Dartmouth College in 1810, by Harvard University in 1825; and the degree of LL. D. was conferred on him by St. Johns College, Maryland, in 1830; by Kenyon College, Ohio, in 1846; by Dartmouth College, New Hampshire, in 1846, and by Harvard University in 1857.

He was a member of the American Academy of Arts and Sciences, of the American Philosophical Society, and of various scientific associations.

The Role of the Engineer Battalion With An Infantry Division

BY

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I. INTRODUCTION.

The infantry division referred to in the subject of this paper is understood to be a team or machine of which the parts cooperate in subordination to the controlling will. Of this machine the engineer battalion is an important auxiliary. To fix its rôle one must clearly understand the proposed action of the principal arms. This, indefinite or changing theory, and peace conditions, have made not always easy. This difficulty is multiplied because, as will appear, engineer duties in any campaign are very diverse in character, and again because they vary more with the character of the campaign than is the case with any other arm or service. Further, when, as is probable with our volunteers, the infantry division falls short of being a perfect machine, the duties of the engineers are still more affected. In short, the rôle of the engineers is varied and complex, so much so that it has been despairingly put thus: "The engineers must be prepared to do anything."

In seeking a better definition, one turns to our laws and regulations only to find them, while correct in the main, very general in expression and somewhat vague and obsolete. Those of foreign nations appear to be little better. In 1908, the Commander of a British Royal Engineer field company wrote: "It was not only necessary to devise the best method of utilizing the unit, but it was also necessary to get the General to adopt it and embody it in his orders. . . . At present a field company Commander's proposals embody his personal opinions only, instead of properly laid down general principles."

When that was written, not only were regulations concerning the employment of the engineers of the modern infantry division defective, but, as far as I have been able to discover, no adequate discussion of the matter had been published in any language.

Since 1908, however, and doubtless following study of the possibilities of engineer troops as shown in the Russo-Japanese War, there has been very extensive discussion, and test in maneuvers, of engineer questions, the French especially going into these subjects very fully.

In studying the writings of continental officers, however, it is necessary to remember constantly the vast difference between their conditions and our own. The same is true of study of the Franco-German War. Because of similarity of conditions we can learn more from the British, whose engineers are probably unsurpassed in Europe. The Russo-Japanese War, because of recent date and up-to-date equipment and tactics, furnishes excellent examples of the best modern use of engineers. A study of our own Civil War is also valuable because the terrain, the combatants and the volunteer system are peculiarly our own. The field engineer features of the Civil War are extremely hard to study because the most important engineer officers, especially General Duane, Chief Engineer of the Army of the Potomac, made practically no reports. Recently, however, Captains Cheney and Ralston have brought to light some of the desired information in a resumé of the duties of engineer officers and troops in the Civil War.

About a year and a half ago the writer began collecting from all sources data as to historical or approved employment of engineer troops. These, revised to allow for differences in conditions, appear in the lists in your hands. (Tables 1, 2, 3, 4, and 5 below.) From a study of these the writer has reached the following definition of the Rôle of the Divisional Engineers.

II. THE ROLE DEFINED.

Divisional engineers are auxiliary troops. Their technical equipment and training should serve one purpose, that of improving the conditions under which the Commander and his principal arms operate against the enemy.

Their work should improve the conditions affecting: 1. Command and supply, 2. Maneuver, 3. Fire action.

Under "command and supply" they assist the General and his Chief of Staff by reconnaissance of terrain and hostile positions, by topographical work of all sorts, and by assisting in devising and securing the most effective use of the engineering resources of the command and of the terrain.

Their work affecting maneuver consists of demolitions and works of communication.

In the domain of fire action and effect, their assistance is brought to bear on the specially difficult or technical features of field fortification undertaken by other troops.

The employment of technical work in field operations is on the increase. General Kuropatkin says: "The great development of science in warfare is very marked, but the late war did not display the employment of scientific forces that will be made in a struggle between two European powers." (The Russian Army and the Japanese War, Vol. II, p. 141.)

III. DUTIES OF ENGINEERS IN THE VARIOUS PHASES OF CAMPAIGN.

In taking up the duties of the engineers in detail, I can only refer to a part of these mentioned in the lists in your hands. These duties will be discussed in the following order, which is, in my opinion, their order of importance: 1. Duties in Attack, 2. Defense, 3. On the March, 4. Retreat, and 5. Camps.

This order, while reversed chronologically, is the order in which any such problem should be considered; that is, first the requirements at the crises, the ultimate mission, and then the requirements in the preparatory phases.

1. DUTIES OF ENGINEERS IN ATTACK.

A. To perform staff duties relating to information, plans for technical work and technical supply.
Including:
1. Reconnaissance with the commander or his representative.
2. Outpost or place sketching.
3. Position sketching (of attackers' position in long-drawn fights).
4. Map reproduction.
5. Photography.
6. Reconnaissance for technical purposes.
7. Projects for employment of the engineer resources of the command and terrain.
8. Requisition of tools and materials.
9. Supply of engineer equipment.

B. To facilitate movement of friendly troops and hinder that of the enemy.
Including:
1. Clearing, constructing, improving, marking and illuminating roads, forks, and bridges (enumerated in detail under "Defense")
2. Ferrying.
3. Assist the cavalry on flanks or hostile rear, by improving or destroying communications, by destroying stores, etc., according to rôle of cavalry.
4. Clearing for artillery observation or fire.

C. To assist in the close attack of fortified positions.
Including:
1. Destroying or providing for the passage of obstacles of every description.
2. Destruction of land mines.
3. Destruction of casemates and other accessory defenses.
4. Sapping and mining (rare in field battles).
5. Use of grenades and high explosives generally.
6. Searchlights and other means of illumination. (Also in fording river.)

D. To provide for succeeding stages of the operation.
Including:
1. Construction of works to protect flanks against counter-attack.
2. Construction of supporting points in rear to check pursuit in case of reverse.
3. Strengthening captured positions.
4. Providing communications to facilitate use of reserves, to facilitate pursuit or withdrawal.

E. Fighting as infantry (in emergencies).

The rôle of the engineers in the attack has developed strikingly. The intensity of the modern fire fight has slowed down the attack, forced the infantry to use the spade, and increased the work of the engineers. Maj. A. Collon, French Artillery, in an article on "The Cooperation of the Arms on the Field of Battle" (Rev. Mil. Gen., 1911, p. 429), says: "In all armies, the number of tools of the infantry and the importance of the pioneer have been increased; in fact, doubled. The Japanese, without relaxation, associated the spade with all their movements on the field of battle."

On the extensive battlefields of to-day, the time elapsing from first close contact to decision may be days or weeks. At the points of long-continued contact, every device will be used to gain advantage or prolong resistance. The result is that all infantry divisions, and especially their engineers, must be ready for close attack or defense of strong field positions, with engineer features resembling in many respects those of siege operations.

Of those duties listed under Attack, section C (table above), sapping and mining, although such work was begun at Cold Harbor and on the Sha Ho, will certainly be rare in field battles. The use of grenades, both hand and large, will probably be common. Just before Mukden several battalions of engineers, made available by the fall of Port Arthur, were distributed among the four Japanese armies as grenade throwers.

Searchlights for use in field battles present an unsolved problem. Our own tests of their tactical value in maneuvers are of no dependence because, though our lights are excellent, the simplest of the requirements, laid down by European experts for their successful use, could not be complied with. (Royal Engineers Journal, Vol. IX, p. 81, and Vol. X, pp. 135, 317.) The Germans have used them with success in the maneuver of forcing a river line, and the great powers appear to be adopting or testing field searchlight companies, with six or more lights, for infantry divisions.

An instance of other kinds of illumination was the use, in the struggle for Manyuyama Hill, at Liaoyang, of strings of burning magnesium balls. (British Officers' Reports, Vol. III, p. 400.)

The most important use of the engineers in attack is at the head of attacking columns. Major Collon, continuing his remarks above quoted, says: "The Japanese pioneers played a capital rôle in the attacks; they were charged with preparing the assault by destruction of the Russian accessory defenses; the execution of this task was often decisive." (Rev. Mil. Generale, 1911, p. 429.) It must

not be imagined that this work can be executed by any men unsupported and without protection. Cover may be afforded by darkness or thick weather, or, as suggested by Colonel Haldane, British General Staff, by having the artillery, with smoke-making projectiles, establish a smoke screen. He instances a case where, on dry ground, the same result was secured from dust raised by the artillery. The use of steel body shields is well known. Unless the work is attempted by stealth, however, these protective measures must be made effective by vigorous fire of infantry and artillery to keep down the defenders' fire.

While this dogged fighting is going on along the line of hostile contact, in rear and on the flanks, skillful and rapid maneuver will often be deciding the battle.

The late General Langlois, of the French Army, says (Rev. Mil. Generale, May-June, 1911, p. 675): "I have long been convinced that the principal mission of engineer troops is to assure communications. In most cases this mission must be carried out quickly; for everywhere and always in modern war speed is an essential factor of success."

The pioneer duties given under section B of "Duties in Attack" need little comment, their importance is obvious. In deployment for attack they will often have to be provided for by assigning engineers to each column. If possible, however, the necessary work should be foreseen and executed in advance, both for attacking columns and for subsequent movements of reserves. In his excellent study of the "Use of Engineers in Cooperation with the Other Arms," Captain Camut, of the French Engineers (Journal des Sc. Mil., Jan.-Feb., 1910, p. 205), says: "In combat it is necessary to assure communications to the rear quite as much for maneuver, movement of reserves or possible retreat, as for supply and evacuation. It is, however, not when the battle has begun, and the need of the engineers with the fighting troops has made itself felt, that engineers should be sent back for this pioneer work in rear; it is rather during the march itself that they should establish these communications. This is only a question of organization and foresight."

In addition to general pioneer work and that at heads of infantry columns, foreign authorities assign to engineers work for the artillery, not only to assist its movements, but especially, by rapid work, to permit it to take advantage of positions otherwise not usable. A particular case of this work is felling trees which would either in-

terfere with observation or cause premature burst of shrapnel. (R. E. Journal, Aug., 1911, pp. 116, 118; Journal des Sc. Mil., Jan.-Feb., 1910, p. 315; Genie en Campagne, Captain Winkler, p. 35.)

The questions of bridges and mounted engineers will be referred to later.

It is not to be understood that the engineers have a monopoly of pioneer work, quite the reverse. Pioneer work is, however, one of their specialties and they should be fully used for it, to save time and strength of other troops and to get the best results in difficult or urgent cases. Even if it is sometimes necessary to reinforce them by details from the line during the heavier part of extensive construction, still, in this work of general interest, the engineers should be made responsible for results and should see to the completion and maintenance of the works.

Turning now to the staff duties of engineers, one sees at a glance that the development of military art and modern science, far from reducing their importance, has only increased the importance of anticipating, and promptly and effectively performing, every staff duty.

It is by no means always that an engineer officer is needed to accompany the commander or his representative on reconnaissance. It is, however, a safe rule that, when the commander rides forward to reconnoiter prior to issuing orders for combat, he should take with him his artillery, signal, and engineer commanders. As to delegated reconnaissances, General Heath, of the Royal Engineers, says (R. E. Journal, Aug., 1911, p. 112): "Engineer officers should always accompany the general staff reconnaissance of a position or of a river or pass, or one made preparatory to a march or for the purpose of selecting billets, camps, etc. The duty of the engineer officer will be to study the best method in which the engineers may be employed, so as to enable the commander of the engineers to advise on technical matters."

Reconnaissance for river crossing should especially have in view the necessity of harmonizing technical and tactical requirements. The best points for ferrying are often not suitable for bridging. Again, the ponton bridge should, as soon as possible, be released by construction of a permanent bridge, and the location of the ponton bridge should be chosen to allow this.

Special engineer reconnaissance for tactical purposes may also be necessary. On this point Colonel Polyanski, of the Russian Army, says (R. E. Journal, Aug., 1911, p. 114): "Reconnaissance

or scouting in the attack is, properly speaking, a duty which belongs to cavalry, but such are the conditions of modern war that it has become impossible for the cavalry to deal with all questions which have to be answered before the plan of attack can be decided upon. The modern arrangements for the fortification of a position are so complicated and reconnaissance is rendered so difficult by means of masking and the use of dummy works, that none but engineer specialists can understand from long distances the intention and character of the various fieldworks which the enemy may have added to the strength of his position.

"That special engineer reconnaissance is necessary first, because evident during the Russo-Japanese War. In the month of September, 1904, when attack operations were in contemplation, it was decided to form engineer reconnaissance detachments, and these were recruited from among the officers of the engineer and sapper units." That such reconnaissance is advantageous is a perhaps forgotten lesson of our Mexican War, in which General Scott largely based his actions on the information of terrain and hostile positions furnished by his engineers, Lee, Beauregard, McClellan, Tower, and G. W. Smith.

As an aid to reconnaissance, photography has not yet received enough attention in our service. It will, of course, have its obvious part in map reproduction, technical reconnaissance and record work.

Closely associated with tactical reconnaissance is topography, a subject which, being very tangible, has probably been given undue prominence in our service at large, justifying the sarcasm in General Weston's remark, "Every time I see a man go past my office with a bundle of blueprints under his arm, I feel safer." Nevertheless, both as training and to facilitate operations in an unmapped terrain, topography is important and our Army's skill in it will doubtless be a great advantage. Topography in new country is, however, very exacting work, and I feel safe in saying that with our trained officers thoroughly occupied in handling their recruits or volunteers, the chiefs of staff will rejoice to realize that their engineer battalions include competent mapping detachments, for which they will only have to outline what is wanted, without necessity of wasting their time or that of their assistants on supervision of purely technical work.

The British Army, which alone of European armies, is prepared for service on other than predetermined and thoroughly mapped

terrain, provides for execution of limited topographical work by the R. E. field companies, and for more extensive work by the R. E. survey companies.

The Japanese engineers in Manchuria did the sketching in front of the army, including position sketching, while civilians surveyed in rear. (Reports of Observers, Russo-Japanese War, Part III, Kuhn, pp. 36, 44.) This is our probable arrangement. In it the engineers act as the technical agents of the general staff information division, which receives and issues all maps so produced. This arrangement, which is strictly in accord with our regulations, has, since 1908, had extensive practical test in the Philippines, and I am informed by officers in close touch with each Manila office concerned—that is, the Military Information Division and the Chief Engineer's Office, that it is decidedly satisfactory to all. Each office is busy, and the Information Division is glad to be free of the burden of directing and supplying surveying parties, while the Chief Engineer is glad to have full responsibility for custody and issue of maps placed where it belongs—that is, on the General Staff.

In addition to the several kinds of reconnaissances mentioned, there is another variety not often considered—that is, engineer reconnaissance for strictly technical purposes, to ascertain the character of the work to be done by the engineers and to find out what tools and materials can be collected from the country for use in the work. This reconnaissance will sometimes be included in that of the engineer accompanying the commander or general staff officer, but often the necessity of having more detailed information will call for additional technical reconnaissance. This will be referred to again under "Duties on the March."

The inadequacy of the proposed portable entrenching equipment for our infantry, one digging tool per two men, is clearly seen in the discussion of the execution of field fortifications in Chapter VI of Colonel Kuhn's "Notes on Field Fortifications." This deficiency will undoubtedly be greatly increased in field service because of loss, wear, and breakage. The Russians found it impossible to resupply entrenching equipment rapidly enough (Mil. Wochenglatt, March 10-12, 1911) and this difficulty of replacement may partially account for the increase of portable tools to one per man in the Russian, French, and Japanese armies.

Although portable equipment is supplied by the Ordnance Department, any deficiencies in it must be met by supply of large

tools from the engineer parks and by requisition. The equipment of the engineer companies should not, except in great emergency, be used to meet such needs, as it is necssary to efficient work by the engineers.

As the attack proper begins the engineers drop behind the deploying and advancing infantry, and are available for further use. This may be to follow the infantry, prepared for work in the close attack. If not required there the engineers should be used to provide for succeeding stages of the operation. The measures noted in the list, Section D, do not receive much consideration in peace study of tactical situations, which are inevitably somewhat lacking in continuity. Such measures will, however, often be made use of by those commanders who "aim to secure to themselves every possible advantage." In case of reverse a line clearly indicated by a few works, the holding of which affects the honor of the troops, has greatest physical and moral value. (Colonel Bell, R. E., Jour. R. U. S. I., Dec., 1897.) On the other hand, effective preparation for pursuit may permit completion of victory. Except when needed in the final stages of attack or as a last reserve, the engineers are especially suitable for this work of preparing for succeeding stages of the operation.

The preceding remarks on the subject of engineer duties in attack largely cover the corresponding work in the other phases, and only a few points pertaining to the other phases need be mentioned.

2. DUTIES OF ENGINEERS IN DEFENSE.

A. To perform staff duties relating to information, planning and execution of technical work, and technical supply.

Including:

1. Reconnaissance with the commander or his representative.
2. Position or outpost sketching.
3. Map reproduction.
4. Reconnaissance for technical purposes.
5. Project for the employment of the engineer resources of the command.
6. Assisting in preparing defensive schemes.
7. Requisition of tools and materials.
8. Supply of engineer equipment.
9. Assisting in organizing working parties of the line. (Exceptional.)
10. Supervising technical work of line troops (rare) or of civilian labor.
11. Coordination of technical features of defense.

B. To facilitate communication (see D below).

Including:

1. Demolition of obstructions to communication.
2. Improving roads and fords.
3. Laying out and constructing roads and defiladed routes.
4. Clearing routes through woods.
5. Marking and illuminating routes.
6. Repairs to bridges.
7. Laying ponton bridges.
8. Improvising bridges.

C. To assist in work usually performed by engineer troops from the rear.

Including:

1. Repairs to railway.
2. Laying field railway.
3. Laying ponton bridge (see B above).

4. Searchlights and other means of illumination.
5. Mining (rare).

D. To assist other troops in preparation and maintenance of defenses to which assigned. (Difficult or technical features, or where specially rapid work is required.)

Including:

1. Clearing foreground.
2. Obstacles.
3. Inundations.
4. Marking ranges.
5. Concealment.
6. False trenches.
7. Supporting points.
8. Blockhouses.
9. Observation stations.
10. Organizing woods.
11. Organizing villages, walls, etc.
12. Flanking defenses.
13. Overhead cover.
14. Bomb and splinter proofs.
15. Local communications.
16. Preparation of grenades.
17. Preparation of mines and fougasses.
18. Assisting in entrenching by use of high explosives.
19. Repairs to works damaged by attack.
20. Clearing for artillery observation or fire.

E. Fighting (in emergencies).

Including:

1. As infantry.
2. With high-explosive grenades.

F. To provide for succeeding stages of the operation.

Including:

1. Communications.
2. Rallying positions.

Defense may include any defensive operations from the temporary defensive to which a portion of the attack will often be forced, up to a defensive so scientifically prepared "that a minimum number of men are required for the defense, allowing of a maximum for the knock-down blow, the counterstroke." (R. E. Journal, Aug., 1911, p. 118.)

In the hasty preparation of a defensive position "it is now accepted as a principle that all field works must be constructed by the troops who are to defend them. . . . The need for hasty works under present-day conditions is so frequent and so extensive that their construction can not be delegated, as was formerly the case, to the engineers, whose numbers will be quite inadequate for such a purpose. There are, however, many classes of work required in the organization of a position which demand operations for which the other troops are neither trained nor equipped and which can best be carried out by technical troops." (Notes on Field Fortification, Kuhn, p. 35.) The numerous items listed under D, "Duties in Defense," will in almost any case include some which, because of peculiar difficulties or urgency, call for assignment of engineers to assist the infantry or artillery.

There is, however, the work on general communications, B and C in the lists, which though less striking than the fortification of the front line, has often quite as great importance. It is more likely to be neglected and will therefore often be advantageously entrusted to the general utility troops, the engineers (see Rev. Mil. Gener-

ale, May-June, 1911, p. 555), assisted, if necessary, by details of other troops. In the question of assignment of engineers, as in other military art questions, a normal arrangement is undesirable. To get the best results the dispositions must be made according to the special situation.

Of the items listed under B, C, and D, only one calls for remarks, namely, item 18, assisting in entrenching by use of high explosives. As to this it is stated (PROFESSIONAL MEMOIRS, March-April, 1912) that the Battalion of Engineers at Washington Barracks secured very promising results with rack-a-rock during last summer's instruction in entrenching at Fort Foote and will continue to test it.

Engineer staff duties in preparation for defense are usually more extensive than in attack. Besides reconnaissance, mapping and engineer supply, in deliberate defense, the engineers may be charged with the entire preparation and execution of the defensive scheme, with, however, one important condition. This condition is that the engineer must either be thoroughly informed as to the tactical and strategical situation and the commander's views and intentions, or he must be provided with this information through the presence of a properly informed representative of the commander, usually a general staff officer.

In defensive organization of positions by troops designated to defend them the assistance that can be rendered by the engineers in planning, coordinating, and supervising will vary greatly. In this connection it can only be stated that to get the maximum strength in a defensive organization is a specialty of engineers.

As to necessity for such work General Heath says (R. E. Journal, August, 1911, p. 118): "Care must be taken to coordinate work and add those accessions so necessary to a properly defined position. Judging from what I have seen, I am sure that much more care should be taken in coordinating work than has usually been the case. I have seen one battalion commander take the top of the crest whilst his neighbor took the bottom of the slopes. I have seen trenches from neighboring sections arranged so as to fire *into* one another, but I have not often seen one section commander arrange his trenches so as to *support* his neighbor. For proper coordination you want a well-thought-out plan." The engineers make no pretense to having a monopoly of this work, but their assistance will be available to whatever extent the commander may find it profitable to use it.

3. DUTIES OF ENGINEERS ON THE MARCH.

A. To assist in gaining information.
Including:
1. Reconnaissance for routes of communication.
2. Surveying and road sketching.
3. Photography.
4. Map reproducing.
5. Reconnaissance with the commander or his representative.
6. Reconnaissance for technical purposes.

B. To facilitate the march.
Including:
1. Demolition of obstructions on roads, fords, or other routes.
2. Clearing routes through woods.
3. Laying out, constructing, repairing and improving roads.
4. Marking and illuminating routes.
5. Improving fords.
6. Preparing approaches to fords and bridges.
7. Repairing bridges.
8. Constructing improvised bridges.
9. Laying ponton bridges.
10. Assisting artillery and wagons through difficult places.

C. To assist in work usually performed by engineer troops from the rear.
Including:
1. Minor repairs to railway.
2. Laying portable field railway.
3. Erecting portable bridges.
4. Laying ponton bridges (see B).
5. Maintaining and providing technical protection for ponton bridges.
6. Ferrying.
7. Strengthening communications to take heavy traffic.

D. To assist in advance guard action.
Including many of the duties given under "Attack" and "Defense."

E. To stand prepared for succeeding stages of the operation.
Including provision and proper disposition of all engineer resources required by the situation and orders.

I have already mentioned engineer duties on the battlefield. For those the engineers must always stand ready while on the march. In addition, they must on the march perform important work. This work is not, however, what fixes the necessary strength of the engineers. On the contrary, the recent increase in strength of the divisional engineers in several great armies were made to meet the demands of the battlefield, not those of the march. Nevertheless, the proper employment of engineers on the march presents problems that are by no means simple.

The contributions of the engineers to the department of information have already been given in general. For the march there are, however, special features that must be taken into account. General Heath says (R. E. Journal, Aug., 1911, p. 112) : "It is important that reports giving details of engineering work required should be sent from the cavalry to the army in rear, from the advance guard to the main body, and from the army to the lines of communications. It is only by early receipt of such information that the engineers with each portion of the force will be able to carry out the necessary work efficiently and promptly." Captain Camut says (Jour. des Sc. Mil., Jan.-Feb., 1910, p. 90) : "There should be an officer with the advance guard cavalry. The march by bounds will give this officer time to reconnoiter obstructions which the col-

umn may encounter and to seek out the resources of the country which may eventually be utilized. This will include communications, obstructions, positions, and resources. Oriented as to the situation, and accompanying a general staff officer if possible, this engineer officer should look with the engineer's eye at features which the general staff officer's and the engineer's own tactical sense indicate as of probable importance, and should study how best to use the natural features and how to improve them for the use of the command.'' Of this same sort of reconnaissance, Major Cambier, whose battalion of engineers, as an integral part of the French XX Corps, has been constantly trained in close association with the line, says (Rev. Mil. Generale, May-June, 1911, pp. 651-3): ''These missions will often fall to officers of our arm in war.'' Further, Captain Normand says (Rev. du Genie Mil., Vol. XLI, p. 213): ''If the officer charged with the reconnaissance judges that the work is indispensable and urgent, he ought not to hesitate to initiate it, if possible, with local labor and material.''

As to road sketching in front, the writer voices the views of several experienced officers in saying that, to augment the sketching by their officers, engineer soldiers can be depended upon to make entirely satisfactory road sketches, provided only that contouring be not required.

The necessity of not only preparing roads, but of maintaining them in condition during the passage of the troops and trains, is obvious. An interesting instance of this work occurred in the Franco-German War. The fortress of Bitsch barred the advance of the II Bavarian Corps. During the 8th and the morning of the 9th of August, 1870, the engineers constructed a road 5½ miles long around the fortress, through a country thickly wooded and mountainous. The engineers were afterward distributed throughout the length of the new road and, by great exertions and by using the horses of the engineer train, were able to prevent all stoppages. (Operations of the German Engineers, Captain Goetze, pp. 41-42.)

As to marking routes, the suggestion that colored cards or lights be used to differentiate routes, seems to offer another proper activity for the engineers.

Bridging operations form a very important part of engineer duties on the march. A partial count of bridges constructed during the Franco-German War, on the march to Paris, gives 123, of which 8 were over the Moselle. (Colonel Kuhn, Staff Class Lecture,

April 15, 1912.) The following are the principal points deserving notice regarding bridging operations:

1. Approaches often require more time than the bridge itself, especially with ponton bridges.

2. Except for single wooden spans across narrow streams, ponton bridges can usually be laid in from one-fifth to one-tenth of the time required for improvised bridges, sometimes much less.

Because of this advantage, foreign armies provide as part of their infantry divisions the following lengths of ponton material for wagon bridge:

England, 120 feet; Germany, 114 feet; Italy, 132 feet; Russia, 70 feet; Japan, 475 feet. Such provision is being strongly urged in France.

All the powers mentioned, except Japan, have also heavy corps (in England, general) bridge trains.

3. Every ponton bridge requires for its maintenance one or two squads, as well as other squads to protect it from destruction by floating devices of the enemy.

4. To use the full capacity of a road, two bridges should be provided.

5. Ponton equipage should be released as soon as possible by construction of an improvised bridge nearby. This work should, if practicable, begin during the laying of the ponton bridge.

6. The work of finishing improvised bridges or strengthening bridges to take heavy traffic should not prevent the divisional engineers going forward with the troops. The heavier bridge work should be taken up by engineers belonging to army headquarters.

This illustrates a good general policy, expressed by a Royal Engineer officer as follows: "The R. E. must not wait to finish the work they start, but must move on with the troops, leaving its completion to the R. E. in rear."

Some of the duties mentioned under C, "Duties on the March," appear to belong to lines of communications troops. They may, however, in the course of events, fall to our divisional engineers in considerable amount, for it must be remembered that our infantry divisions may be sent on independent expeditions, as in the Spanish-American War, without being supplied with engineers for lines of communications work. In the Philippines the engineers were called on not only to repair the railway but, ultimately, even to run it. It is to be hoped that our reorganization will provide for such cases

in such a way as to leave the divisional engineers to their proper work.

4. DUTIES OF THE ENGINEERS IN RETREAT.

The duties of the engineers in retreat include many of those listed under "Defense" and under "Advance," and, especially, demolition and obstruction of communications, and destruction of material and stores.

5. DUTIES OF ENGINEERS IN CAMP.

A. To prepare the camp.
Including:
1. Selecting sites.
2. Laying out the camp.
3. Preparing plat of camp.

B. To facilitate communication.
Including:
1. Clearing, constructing, improving, marking, and illuminating roads and bridges.
2. Preparing ramps to watering places.
3. Working back along the lines of communication during periods of certain inactivity.

C. To perform, or assist in work usually pertaining to other departments or units, but for which they can not always be prepared.
Including:
1. Establishing water supply. (With advice of medical officers.)
2. Marking watering places.
3. Preparing watering troughs.
4. Assist in sanitary constructions requiring technical skill.
5. Constructing shelters for permanent camps.
6. Constructing landing facilities.
7. Constructing entraining and detraining facilities.

D. To assist in instruction in field fortifications and other engineer work.

Of engineer duties in camp, these points should be noticed:

1. In the vicinity of the enemy, the situation will frequently call for defensive organization of the outpost line, and for preparation of interior communications to facilitate deployment in any direction. Technical work so applied may have decisive value. At least, it should materially reduce the strength necessary for the outpost.

2. Water supply often becomes a serious question in campaign. Normally under the Q. M. D., in emergency it is usually handled by the engineers. On the march, however, troops must generally establish their own supply. In the British service, which alone is prepared for campaign in arid regions, the troops are furnished with water carts and the R. E. with each infantry division have four large troughs and four pumping outfits.

3. Engineers should not be equipped or trained for the miscellaneous mechanical work of camp or garrison. Their purpose is to render service on the march and on the field of battle. As one of our officers has put it: "If the commanding general sends to the engineers to have a box made for his own use, he should get the box; but, with engineers properly equipped, it should be of rough lumber and 10d. nails."

IV. DISTRIBUTION AND DIRECTION.

The extensive duties outlined in this paper are, if taken en masse, obviously beyond the powers of a single engineer battalion. The strength of engineers is always a compromise. In preparing a paper on that subject (''The Proper Organization and Equipment of a Field Battalion of Engineers'') a year ago the writer concluded that, for the strictly pioneer portion of our battalion, the present strength is approximately correct, but that 100 to 200 specialists are needed to avoid frittering away the pioneer companies by special duty details.

That is a question pertaining to the subject of organization. For the commander in the field, the practical problem, in regard to the engineers, is how to make the most effective use of the engineer resources provided. This will be discussed as to (1) how the orders should be issued, and as to (2) distribution on the battlefield, and (3) distribution in column.

1. ORDERS.

In 1898, Captain Krisak of the German infantry said (quoted in Rev. du Genie, Vol. XXXVII, p. 30): ''The sure choice of the works which it is necessary and possible to execute in each particular case, and, especially, the art of giving to technical troops orders in the closest conformity with tactical requirements, calls for skilfulness which is not sufficiently widespread.''

Since that was written much more attention has been paid to technical features of operations. The general result has been to throw on the senior officer in each technical branch more and more the responsibility for proper distribution and employment of his units to accomplish specified purposes of the commander. To quote Major Colon, French Artillery (Rev. Mil. Generale, Vol. IX, p. 429): ''In order that the pioneers may render the services expected of them it is necessary to specify their mission concisely. It follows that the commander of engineers attached to a unit should receive information to orient him as to the general situation, intentions of the commander and purpose of the work ordered. This condition being satisfied, it ought to be well understood that tactical requirements take precedence over technical necessities.''

An approved method (General Heath, R. E. Journal, Aug., 1911, p. 113; Theme de Fortification, Lenoble, p. 36 et seq.; Genie en Campagne, Normand, p. 94) is to call on the senior engineer for a

project for the employment of his resources, based on his special reconnaissances, or to have him review the portions of a general project which affects his arm. The only objection to this is that, being a commander of troops, he may not be at hand to consult. (Rev. Mil. Generale, May-June, 1911, p. 561.) With our organization, however, the engineers will only exceptionally work by battalion and the commander will usually be more useful with the division commander. As a matter of fact, in our large divisions corresponding more to the European corps than to European division, the engineer commander is, to all intents and purposes, a chief engineer officer. With this fact in view it has been proposed (Duties of Engineer Troops in a General Engagement of a Mixed Force; Capt. Burgess) to give our battalion commander an assistant or second in command to run the battalion under more or less general orders from the commander, whose post would be at division headquarters.

A chief engineer or the equivalent at headquarters is desirable to give technical advice when required and, from his knowledge of their capacity and limitations, to keep the engineers from being assigned impossible tasks, or, on the other hand, from being entirely forgotten. Also, the chief engineer should keep track of the engineer troops and resources, including any placed under immediate orders of brigade or other commanders, because they may have to be withdrawn and sent to other points. Being oriented as to the situation, the chief engineer should prevent engineers continuing work that has lost prospective value. (Rev. Mil. Generale, May-June, 1911, p. 661.) This may often occur and the men should be trained to understand it, for engineer work is generally preparatory in character and must be begun well in advance. (Jour. des Sc. Mil., Jan.-Feb., 1910, p. 206.) Really premature dispositions are, however, to be avoided.

In general, then, it may be said that orders for engineer troops attaching them to other units should usually form part of the orders to those other units, the senior engineer being informed and, if possible, previously consulted. All other orders for engineers, whether formal or informal, should, if practicable, be given through or by the senior engineer, whether he be called chief engineer officer or not.

2. DISTRIBUTION ON THE BATTLEFIELD.

Proper economy prohibits the use of normal, cut and dried, distribution of engineer troops. A British authority says (Heath, R. E. Journal, Aug., 1911, p. 117): "In maneuvers it is often the practice to break up a field company into sections and distribute

these all along the line . . . a slovenly, sealed-pattern way of doing things. . . . Field companies should be held together until it is decided what work there is for them, and not broken up and distributed all along the position on the chance of there being something for them to do.''

Buddecke (in ''Tactical Decisions and Orders'') says: ''As a rule, the engineers should not be split up in working. They can be used to better advantage at important points and in the execution of important tasks by company, platoon, or section.''

Captain Normand, of the French engineers, gives the following ''principles'' governing the employment of the engineers (Rev. du Genie, Vol. XLI, p. 193 et seq.):

1. Works are generally prepared by those who will be called upon to use them.

2. Engineers are employed: On works of general utility; or, presenting technical difficulty; or, requiring special equipment; or, requiring special speed of construction, and in organizing points of support of capital importance.

3. A company of engineers always remains as concentrated as possible, under the control of its officers.

4. The company parks of division and corps companies should never be separated from them and should not constitute a reserve for the other arms.

5. Corps engineer parks, bridge trains, and army park should be detached toward the front, according to the situation and the intentions of the commander.

6. All engineer work should be preceded by reconnaissance in company with the representative of the C. O.

The above rules are believed to represent the best practice. Their importance has been brought out in the discussion of engineer duties, except as to the engineer company's remaining as much concentrated as possible.

In discussing this principle, Captain Normand says: ''This concentration does not, however, require that the engineer company be always worked assembled at one place. A captain of infantry who furnishes an outpost support and a number of pickets still commands his company; the same principle can be applied to the engineers.'' With good organization and means of communication, the distances separating working parties can be quite large without loss of control. The reasons in favor of this unity of command are much the same for the engineers as for the line. Their work will be better technically and tactically, and administration will be

easier, while, if changes in the situation call for changed dispositions, the new demands can be met. Nevertheless, the engineers are with the division to render service and, if this can only be accomplished by dispersion, the disadvantages must be accepted.

3. DISTRIBUTION IN COLUMN.

Distribution in column must have regard to both march and battlefield demands. Again, no normal formations can be used, but certain principles covering march distribution can be laid down.

First, "for duty at heads of columns—that is, with the support of the advance guard, a platoon or a section (120 or 60 men) is usually sufficient, and avoids getting a large number of engineers involved in the action of advance guards" (Rev. Mil. Generale, May-June, 1911, p. 659), and lost for their special mission.

If not so involved, engineers, if placed far forward in the advance guard, would still probably not be in proper position to work for the main body of the column. The reserve of the advance guard should, however, have a proportion of engineers suited to its prospective rôle. These should also see that the road is ready for the main body after the passage of the advance guard.

The remaining dismounted engineers have (1) to maintain the road in condition and assist vehicles in trouble, and (2) to be in readiness for eventualities.

To maintain the road in condition, a small detachment should often be placed at the tail of the main body, and sometimes at other points of the main body. In addition to assisting vehicles, it has been suggested that engineers can serve to place rifles in the middle of a long column of artillery. (Jour. des Sc. Mil., Jan.-Feb., 1910, p. 98.) This is questionable practice.

The balance of the battalion, which should be at least two companies if the division is on one road, in order to be ready for duty in the deployment of the division, should be placed well forward. This will allow detachments to join the *heads* of brigade columns or other detachments to which assigned, and to be in position early for technical work on the battlefield. When the attack begins it is their turn to drop behind. General Heath says (R. E. Journal, Aug., 1911, p. 113): "During the march the greater part, if not all, of your engineers should be with the advance guard . . . if there is work for the engineers, it will be in front and usually, until the engineers have done their work, the column will not

be able to advance. It is easy to get your engineers off the road and out of the way if they are at the front and not wanted there, but it is quite another thing to get them quickly to the front from the rear.'' The engineers seem to have a strong claim to a place in the much-discussed interval between advance guard and main body.

As a principle, preparation of the road should start just as soon as the preceding unit has passed. For this purpose, if the cavalry is in front, mounted engineers, or possibly engineers in wagons, should be immediately behind it. As to engineers to aid in pioneer work for the cavalry in front or on the flanks of the division, this is for the cavalry to decide. All, except one, of a considerable number of cavalry officers whose opinion I have asked, have expressed the view that cavalrymen as pioneers are ineffective, and that a section or so of engineers is needed with the cavalry. Whatever arrangement is made, it seems that all mounted engineers with the division should be united for training, the foot companies having only the mounted men necessary for orderly work.*

The French make a great point of easing the march of the engineers in order to get more work out of them. (Genie en Campagne; Winkler, p. 29 et seq.) They provide (1) an extra wagon per company to carry the packs of engineers doing pioneer work, (2) extensive use of requisitioned wagons to carry engineer packs, extra tools, and personnel; (3) a tool sling entirely separate from the pack.

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*The cavalry referred to on page 794, first new paragraph, is divisional cavalry either in direct contact with the enemy or behind an imperfect screen.

Two classes of work are referred to: (1) Work *behind* the cavalry to get the roads, bridges, etc., into condition for march of the main body. For this engineers, either mounted or in wagons, are required. If mounted engineers are not available for this work, it will usually have to wait until the arrival of the engineers of the infantry advance guard, as the cavalry could hardly handle such work. (2) The other class of work referred to is pioneer work at head of cavalry units to facilitate their march, and demolition work for obvious purposes in front or on flanks of the division. This work may be done by cavalrymen trained as pioneers, or by engineers detailed for the purpose when there is prospective work, or by a combination of the two. If the cavalry can get results from cavalrymen trained as pioneers without interfering with the effectiveness of those men as cavalry, that is the proper solution. This is the point on which numerous cavalry officers have expressed doubt. An arrangement for annually attaching the necessary cavalrymen to engineer organizations for a few months training in pioneer work would obviate much of the difficulty of providing pioneers for the cavalry.

They insist that engineers should rarely be separated from their tool wagons or pack mules. (Genie en Campagne; Normand.)

They further say (Rev. Mil. Generale, May-June, 1911, pp. 653-655), "Engineers must be trained to march as well as infantry, also to maneuver under fire like infantry."

V. CONCLUSION—REORGANIZATION AND TRAINING.

I trust that I have shown that the rôle of the engineer battalion as an auxiliary of the infantry division presents many problems.

The entire subject of the engineer service in the field is now being studied by the recently constituted Board on Engineer Troops. Its instructions from the Chief of Engineers are understood to call for a comprehensive report as to the proper strength, organization, equipment, and duties of every engineer formation.

It is probable that extensive changes will be made in the divisional engineer establishments, particularly in equipment.

It seems quite certain that these changes will include—

A moderate degree of specialization with the primary purpose of stripping the pioneer companies of all personnel and material not absolutely required in the first line.

A great increase in the mobility of the pioneer units and, especially, provision for instantly subdividing them into proper working sections of 40 or perhaps 20 men, each section being provided with its own separate equipment and light transportation, probably a two-wheeled cart.

Given such engineer battalions as I have in mind, the commanders of the mobile divisions into which we hope to see our Army organized should be able to train them to fulfill their proper rôle. Such training should include a number of months of company training, and probably a month of combined engineer training, but all this should be given life by extensive training in combination with the other arms and by association of engineer troops with the other arms in garrison. Thus only can the infantry division, the machine, be ready in all its parts, for field service, and thus only can we be prepared to organize other like machines to meet the emergencies of war.

Economic Material for Boat and Barge Construction*

BY

Mr. A. E. HAGEBOECK
Inspector in Charge of Creosoting Operations
U. S. Engineer Office, Rock Island, Ill.

Our office has been collecting data on the cost of repairs of our standard barge, 100 by 20 by 4 feet 7 inches, for the past twenty years. As we are building the same size barge to-day, the cost of repairs will be directly comparable.

It has been found practical to frame and creosote the timbers in transit at a commercial treating plant, and then forward the timbers to the point of erection. By marking the pieces that can not be easily identified, it is possible to assemble the barge quite rapidly.

In the past five years I have examined a large number of untreated barges at various points on the river that have been in service from four to fifteen years. From these observations I would say that the decay always starts where there is an excess of moisture, together with the air and heat. In 90 per cent of the cases the decay starts in the ends of the timbers. That is to say, the decay develops in the same ratio as the wood absorbs moisture through the ends. As a good pressure treatment will always plug the ends of the timbers, it is easy to understand why such good results have been obtained in the past with a pressure treatment of coal-tar creosote oil.

In former years the opinion was held that it would not pay to creosote a barge because it would wear out before it decayed. This may be true under certain conditions, but as a general proposition I have found the lumber decays first, and when in this decayed condition is easily broken.

For barges used in fresh water it is not considered necessary to creosote the bottom, as it has been found that the bottom plank rarely decays. This fact can probably be attributed to the exclusion of air, as a barge usually contains 4 to 6 inches of water on the inside.

In constructing light-draft barges it has been our policy to use the pressure-creosoted fir, as fir can be obtained in long lengths

*Reprinted from the Report of Proceedings of the Eighth Annual Meeting of the American Wood Preservers Association, Chicago, Ill., 1912.

at a reasonable cost. Long timbers are especially desirable in barge construction, as they reduce the number of gunwale joints to a minimum. The gunwale joints are always the first points to cause trouble by leaking, and so it is a big item to reduce these joints to a minimum. Besides being cheaper in cost, both before and after creosoting, the fir is lighter, resulting in a draft of but 9 inches for a standard barge 100 by 20 by 4 feet 7 inches.

White oak has been used almost exclusively in the past for the construction of model-shaped steamboat hulls. The present tendency is to use steel. Creosoted timber is eliminated from consideration for model type hulls on account of the necessity of framing and cutting timbers during erection, which would expose untreated surfaces if creosoted timbers were used. It is the opinion of the writer that the steel hull will give more economical

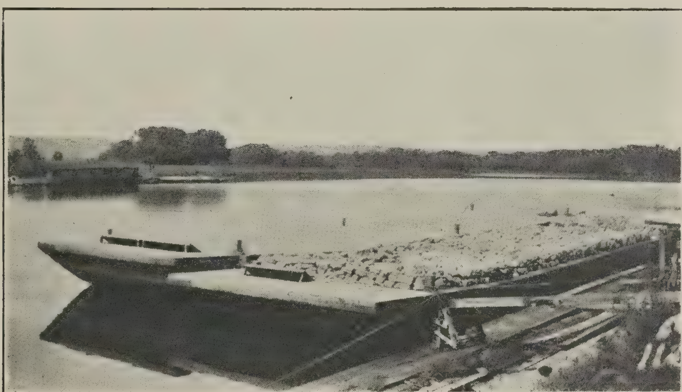

Fig. 1. Two rock barges; size, 100 by 20 by 4 feet 7 inches.

results, for the reason that when the cost of repairs on an untreated oak hull during its life is added to the original cost, the yearly charge will closely approximate that of a steel hull.

These relations, however, do not exist in the case of "scow" pattern boats and barges. A steel barge will cost more than three times as much as an untreated fir barge, and nearly three times as much as a creosoted barge. The lumber for these "scow" pattern boats and barges can be advantageously framed and bored before treatment.

Before taking up the relative costs it will be well to consider a few details.

KINDS OF TREATMENT USED IN BARGE CONSTRUCTION.

The first creosoted barges used in this country were built in 1900 of pressure-treated yellow pine, by the New Orleans office of the United States Engineer Corps. These barges are to-day in a per-

fect state of preservation, and in all probability will be used for ten to twelve years longer. The cost of repairs has been light, and the results so satisfactory that no untreated barges are now built by that office.

The Rock Island District formerly used the open-tank treatment. The penetration was usually superficial, but the cost is only 5 per cent of the total cost of a fir barge. Last fall the writer inspected a large number of these fir barges built in 1908, and in no case was any evidence of decay found on the treated timbers, while in a number of cases the untreated timbers had reached an advanced stage of decay. It is therefore evident that the small cost of this treatment will pay good returns on the money invested. In the case of 90 per cent heart long-leaf pine the same conditions exist, as the penetration on the heart surfaces is usually superficial. With the short-leaf and loblolly pine it has been our experience that this class of timber requires so much oil to saturate the sap that it often costs more than a 10-pound pressure treatment. For treating barge timbers the pressure treatment has, in the opinion of the writer, a number of advantages that make it a far more economical treatment. First, from a treating standpoint, it is possible to treat either green or seasoned lumber. Second, the exact quantity of oil injected can be ascertained by the temperature and gauge readings. Third, the entire treatment can be regulated to meet the requirements of each particular charge. Fourth, it is possible to plug the ends of the timbers and thereby retard the absorption of moisture. Fifth, the penetration of oil is far more uniform. The last two factors tend to eliminate the so-called "working" of the timbers. This is an important item in barge construction, as it is a well-known fact that a barge built of green untreated lumber will usually cause trouble from leaking, due to the subsequent shrinkage of the timber as it dries, and the consequent opening of the seams and loosening of the oakum. Even after the lumber has once become dry it readily absorbs moisture during a wet period and again gives it up during a dry period, and as a result an untreated barge is recalked every year after its fourth or fifth year in service. The pressure treatment has largely eliminated this recalking, and so materially reduced the cost of repairs.

LIFE OF UNTREATED YELLOW PINE BARGES.

On the Mississippi River, between St. Paul and St. Louis, untreated yellow pine has been used but little, and the writer has been unable to obtain any accurate records of its lasting qualities. It is generally believed that unless timber practically free from sap is obtained its life would be exceedingly short. On the Lower Mississippi River a yellow pine untreated barge containing a minimum proportion of sappy timber is past economical repairs at the end of ten years.

LIFE OF PRESSURE-TREATED YELLOW PINE BARGE.

Pressure-treated yellow pine barges have been used on the Lower River for twelve years. These barges are to-day in a perfect state of preservation, and without doubt are good for an additional life of ten years. It has been found necessary to recalk the barges after two years service, but otherwise the repairs have been small, and but little further recalking seems necessary during the life of the barge. One reason for this recalking is that the creosote oil acts on the oakum and ''burns'' it out. As a matter of fact, however, the lumber in these barges was treated while in a green condition, and, in the opinion of the writer, the real necessity for recalking is due to the subsequent shrinkage of the timber and consequent opening of the seams and loosening of the oakum.

On the Lower Mississippi River, where there is always a good stage of water, light draft is not a controlling factor, and so barges 120 by 30 by 6 feet and larger are in general use. The original cost of these untreated yellow pine barges, built in the early 90's, was about $3,000.00; the cost of repairs during the life of ten years averaged $2,006.61 per barge. The original cost of similar barges built of pressure-creosoted yellow pine was $4,000.00; the cost of repairs on ten barges averaged $557.35. The total cost of the untreated barge was $5,100.00, for which nothing remained at the end of ten years to show for this expenditure. The total cost of the creosoted barge during nine years was $4,557.35, and a good barge, now appraised at $3,600.00, remains on hand. The following table shows the comparative annual cost per barge.

Table 1. Comparative annual cost of treated and untreated yellow pine barges. 120 by 30 by 6 feet.

Items.	Untreated barges, ten years old.	Creosoted barges nine years old.
Original cost	$3,093.39	$4,000.00
Cost of repairs	2,006.61	557.35
Total cost	$5,100.00	$4,557.35
Subtracting value of barge to-day	0.00	3,000.00
Total cost of untreated barge during ten years	$5,100.00	
Total cost of creosoted barge during nine years		$937.35
Annual cost per barge	$510.00	$106.00
Annual saving in favor of creosoted barge		$404.00

LIFE OF UNTREATED DOUGLAS FIR BARGE AND COST OF REPAIRS.

With but few exceptions, the necessity for repairs to an untreated barge is due to decay and not to mechanical abrasions. Ordinarily, the decks of barges used for rock transportation will first decay on the bottom side at the points of crossing other timbers, and in this weakened condition are easily broken. Fig. 2 shows a number of deck planks that were replaced because of de-

cay. These plank were ordinarily 2½ inches in thickness, and after eight years service measure 2⅜ inches, but, because of their decayed condition and not because of wear, it was necessary to replace them.

It is not uncommon to find evidence of decay on untreated fir barges after three years service. Fig. 3 shows a fungus growth on the deck of a storeboat hull. This boat has been in service but four years, in the vicinity of Winona, Minn., where the seasons are short, yet it gives conclusive evidence of the rapid decay of untreated woods in this class of construction.

The repair costs used in Fig. 4 have been obtained from the plant records of the United States Engineer Office at Rock Island, Ill., covering Douglas fir barges in use for the past twenty years in connection with the work of improving the Upper Mississippi River,

Fig. 2. Deck plank replaced because of decay.

between St. Paul and St. Louis. In the past, untreated fir barges were kept in service ten to seventeen years, the average life being fifteen years. The diagram is intended to show the usual cost of repairs from year to year during the life of an untreated Douglas fir barge. New barges are, as a rule, used for rip-rack rock transportation, this service requiring a substantial craft. From the diagram it will be noted that during the sixth and seventh years the barges required extensive repairs, the cost ranging from $200 to $300 per barge; that with repairs costing about $75 per year they continued in hard service to the tenth or twelfth year; that they then required large repairs and had to be taken from rock work and placed in the brush carrying service, which is much less severe on account of the large decrease in weight per cubic foot of load. From this time on to the end the cost of repairs per barge is largely increased; and it is debatable whether it would not be fully as economical to abandon the barge at about the tenth to twelfth year.

LIFE OF PRESSURE-TREATED DOUGLAS FIR BARGES AND COST OF REPAIRS.

It seems safe to estimate the life of creosoted fir barges at twenty years, since untreated barges have given an average life of fifteen years. The major portion of the repairs on an untreated barge are for calking and repairs to deck, rake, and gunwale joints, on account of decay. As the present tendency is to air-season the fir before treatment, it seems natural to believe that the barges will give a long service without recalking, as was the case of the creosoted barges used on the Lower River, as cited before. As an additional precaution, it is thought advisable to protect the creosoted deck with a 1-inch wearing surface of untreated material. The repairs to the deck are therefore confined to the occasional relaying of this protection.

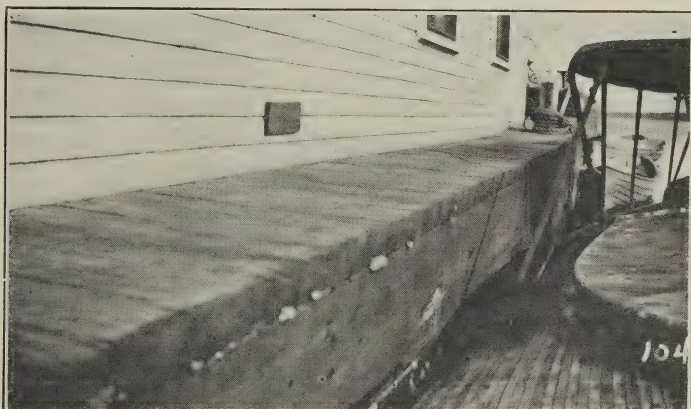

Fig. 3. Untreated Douglas fir hull four years old.

Table 2. Comparative cost of light draft barges built of various kinds of material. Size, 100 by 20 by 4 feet 7 inches.

Items.	Douglas Fir.		Yellow Pine.		Steel.
	Untreated. 15 years life.	Treated 10 lbs. 20 years life.	Untreated. 15 years life.	Treated 14 lbs. 22 years life.	25 years life.
Original cost	$1,200	$1,500	$1,300	$1,650	$4,000
Total repairs	1,094	400	1,094	700	400
Interest at 5 per cent on cost	900	1,500	975	1,815	5,000
Interest at 5 per cent on repairs	341	125	341	125	125
Total cost	$3,535	$3,525	$3,710	$4,290	$9,525
Annual cost per barge	$236	$177	$247	$182	$381
Annual saving in favor of creosoted fir barge	$59	------	$70	$5	$204

Table 2, for timber barges, is based on Government freight rates on timber, and so, for commercial comparison, $10.00 per barge should be added to the yearly cost.

The yearly costs show the following relative order for economical barge construction : Creosoted fir, creosoted yellow pine, untreated fir, untreated yellow pine and steel.

It will be noted that the annual cost of a steel barge is twice that of either a fir or pine creosoted barge. The average cost of repairs on thirty-one fir barges used on the Upper Mississippi River was $1,094 per barge during an average life of fifteen years. The original cost of an untreated barge built to-day would be approximately $1,200. On this basis, with interest at 5 per cent, the cost per untreated fir barge per year would be $236, as compared with $177 yearly cost for a creosoted fir barge, or a difference of $59 per year in favor of the creosoted barge.

CONCLUSION.

In conclusion I will state that the original cost of a steel barge with interest on the investment is not compensated for, by the

Diagram showing cost of repairs and life for untreated Douglas fir barges at three points on the Mississippi River.

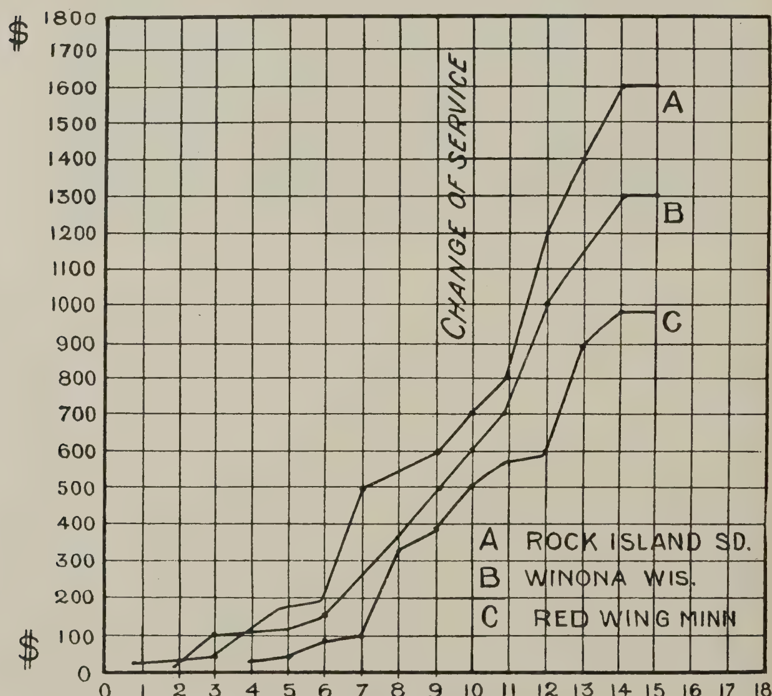

Fig. 4. Years in service.

added life, compared with a creosoted barge. That the cost of repairs on an untreated barge and its short life of real hard service makes the annual cost, including interest, 25 per cent more than for the pressure-creosoted barge. That the pressure-creosoted yellow pine barge and the pressure-creosoted Douglas fir barge have respective fields, depending upon the working conditions; on the Lower Mississippi, where there is always a good stage of water, the creosoted yellow pine will probably be more desirable, but for light draft and use on the Upper Mississippi the pressure-creosoted fir will be far more economical.

* * * * * * *

Mr. J. B. Card: Mr. President, what little experience I have had with Douglas fir I have found it a hard wood to treat, especially 6 by 12 by 40 feet long. It seems to me if I were building barges and the choice lay between Douglas fir and Southern pine, that I would take Southern pine. You can get pretty good penetration on that—much better than you can with Douglas fir. With the Douglas fir we have treated here in this section of the country it has developed in long lengths sometimes the penetration won't exceed an inch in depth along the sides, and I don't see how you can expect to get twenty years life out of a stick of timber that is treated 1 inch in depth with creosote oil. Mr. Hageboeck spoke about the long life in European countries. I don't think that life is obtained from timber creosoted with as little penetration as that. He also mentioned the open-tank treatment. The Government had considerable experience back in the eighties with the Seeley process. I think you will find if you look up the records of the open-tank treatment that within about six years all of that timber was replaced.

Mr. Hageboeck: I agree with Mr. Card that fir is a difficult wood to treat; that time is required to obtain good results, and that the open-tank treatment will not give as economical results as the pressure process. If you are comparing the probable life of treated fir and treated pine, irrespective of the purpose for which it is to be used, I should prefer the treated pine. But for barge construction for work on the Upper Mississippi the creosoted fir is, as I have stated in my paper, cheaper in first cost, both before and after creosoting; it makes a lighter draft barge, resulting in an increased capacity, and so is more economical.

Mr. J. H. Waterman: Where are your barges built?

Mr. Hageboeck: Let me finish my point and then I will answer your question, Mr. Waterman. As I said before, it seems from what information I have obtained that all that is needed is to plug the ends of the timbers, as it has been our experience that the decay always starts where the wood absorbs moisture through the end grain. Now, going back to your question, Mr. Waterman, I will say that we build our barges at these points on the Mississippi River, Fountain City, Wis., eight miles from Winona, and at

Milan, Ill., just a mile and a half from the Mississippi River, on the Hennepin Canal.

* * * * * * *

Mr. P. R. Walsh: Isn't it a fact that when you made a specification for yellow pine that you required 95 per cent heart?

Mr. A. E. Hageboeck: That is just the point I was getting at. If you are not going to treat yellow pine you will have to buy 95 per cent heart, but if you are going to treat under pressure then I say no; then we take all the sap we can get.

Mr. Walsh: You would not need any treatment with 95 per cent heart, would you?

Mr. A. E. Hageboeck: I am inclined to believe you would. On the inside of a barge there is always enough moisture to develop decay.

Mr. Walsh: And it never would rot?

Mr. A. E. Hageboeck: Yes, it would rot.

Editorial Notes

Reprinting No. 2, Volume One

The edition of No. 2, Volume I, has been exhausted nearly a year, and as the publishers of the MEMOIRS have received a large number of inquiries for that number they are considering the advisability of reprinting it.

If all who desire a copy will at once notify the PROFESSIONAL MEMOIRS, the publishers will undertake the reprinting of it as soon as assured of the sale of forty copies. The price will be one dollar, net.

No. 13, Volume Four

The edition of this number is completely exhausted, and the publishers will be glad to pay the full subscription price of fifty cents for all copies in good condition returned to them.

Change in Memoirs

Beginning with this number, there is inaugurated a slight change in the MEMOIRS which, it is believed, will be appreciated by all. This change consists in the following: First, placing the Selected Articles of Engineering Interest at the front of the book and leaving them unpaged, so that in binding they can be omitted without disturbing the paging; second, separating entirely all reading matter and advertisements.

Prizes for Articles for 1913

For articles accepted and published in Nos. 19 to 24, inclusive, comprising Volume V to published in 1913, the PROFESSIONAL MEMOIRS will award four prizes, as follows: Fifty dollars for the best article; twenty-five dollars for the second best; fifteen dollars for the third; and ten dollars for the fourth.

This offer is open to all subscribers to the PROFESSIONAL MEMOIRS, except officers of the Corps of Engineers with more than ten years'

commissioned service in the Corps. The School Board of the Engineer School, which publishes the PROFESSIONAL MEMOIRS, will decide on the best articles and rate them as first, second, etc.

The award for the prize given in Vol. III was made by three Assistant Engineers who were not competitors, and the same will be true of the four prizes to be awarded in Vol. IV. However, a great deal of difficulty has been found in getting three assistant engineers with sufficient time to go over all the articles with the necessary care to decide on their relative merits and to get into correspondence with each other preparatory to agreeing as to which are the best articles. It is therefore deemed best all around that the School Board hereafter award the prizes.

Articles of any length whatever will be considered for all four prizes, though naturally a good article of considerable length will be rated higher than one of relative merit though shorter. Professional excellence, clarity and conciseness are considered of prime importance, though literary excellence will be considered in making the award. In addition to the above prizes and in accordance with the precedent of the past two years, one year's free subscription will be given to each author of an article of twelve pages or more, if that article be not awarded one of the four prizes above mentioned.

Vol. IV NOVEMBER-DECEMBER, 1912 No. 18

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ENGINEER DEPARTMENT AT LARGE

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Compiled by Henry E. Haferkorn, Librarian, Engineer School.

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(38) Kriegstechnische Zeitschrift (Berlin).

(39) The Contractor.

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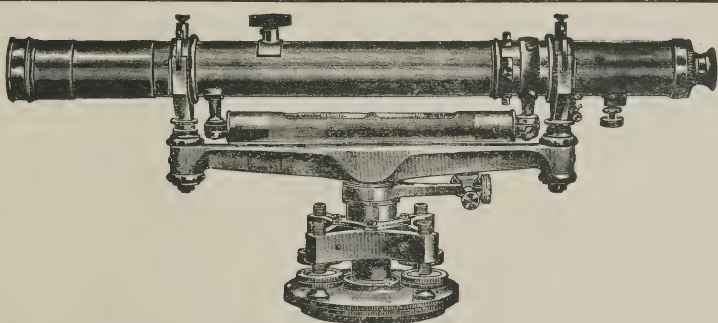

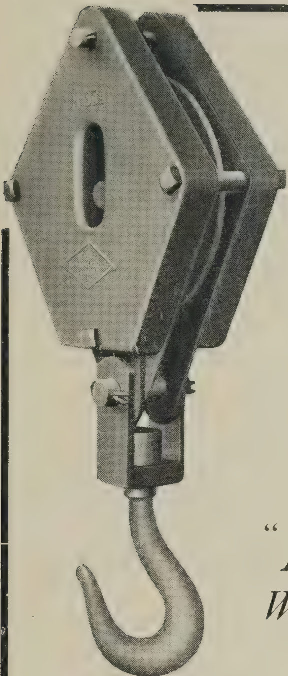

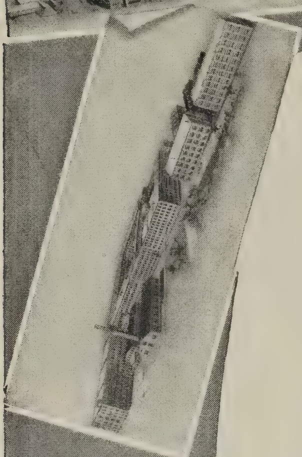

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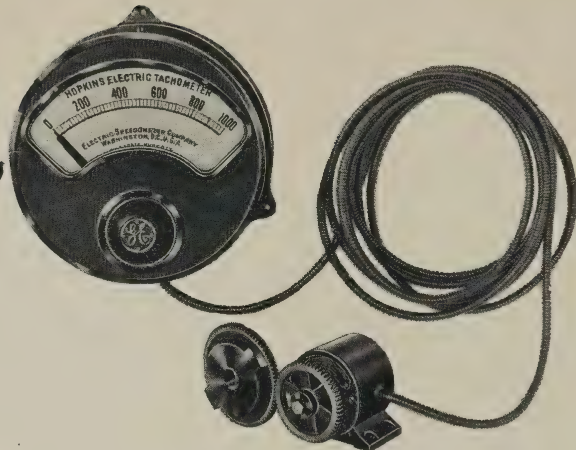

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